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Volume 26

April 1950 - February 1952

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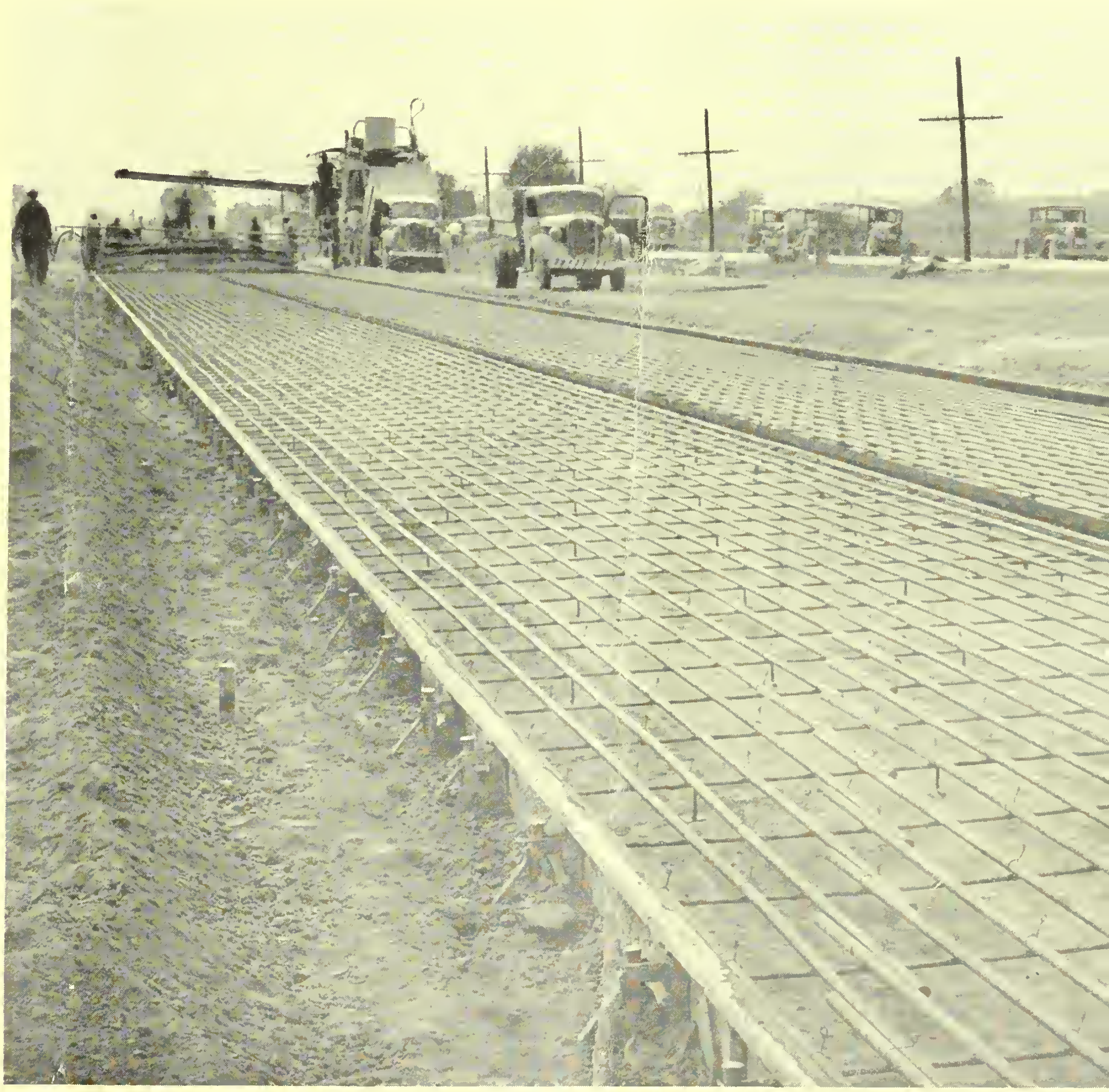

Continuous longitudinal bar reinforcing steel in Indiana experimental concrete pavement

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BUREAU OF PUBLIC ROADS

U. S. DEPARTMENT OF COMMERCE

E. A. STROMBERG, Editor

Contents of this publication may be reprinted. Mention of source is requested.

Continuous Reinforcement in Concrete Pavement

A Cooperative Investigation by the Bureau of Public Roads and the Indiana State Highway Commission

Reported by HARRY D. CASHELL,
Highway Research Engineer,
Bureau of Public Roads,
and SANFORD W. BENHAM,
Research Engineer, Indiana
State Highway Commission

Concrete pavements, unless constructed with closely spaced transverse joints, generally crack transversely at frequent intervals. These cracks tend to open appreciably with time unless the pavement is reinforced. For some years there has been a continued interest on the part of highway engineers regarding the practicability of concrete pavements constructed without transverse joints and reinforced longitudinally with continuous bonded steel in sufficient amount to hold all cracks closed.

In the fall of 1938 a considerable number of continuously reinforced sections, ranging from 20 to 1,310 feet in length, were constructed near Stilesville, Indiana, on U S 40 as a cooperative research project to study the effects of varying amounts of longitudinal steel in sections of various lengths.

The behavior of the sections during the first 10 years of service life conclusively shows that continuous reinforcement can be depended upon to prevent the opening of transverse cracks in concrete pavements. In the long, heavily reinforced sections many fine cracks have developed in the central region. These cracks have not opened and have raveled only slightly with traffic and exposure, a condition that has required no maintenance and may be considered superficial. The sections have remained strong, durable structural units.

The presence of even the heaviest longitudinal bar reinforcement has apparently not affected adversely the condition of the concrete in the pavement. The concrete appears to be sound throughout, there has been no spalling, and there is a complete absence of longitudinal cracking above the bars. In fact, the manner in which the steel has held closed all cracks, especially those in the more heavily reinforced sections, is believed to have been conducive to distributed interfacial pressure at the cracks which should tend to minimize damage to the concrete from concentrations of pressure such as sometimes develop at cracks in plain concrete pavements.

Pumping has developed at many of the transverse joints but, with two exceptions, has not been observed at any of the vast number of transverse cracks. This indicates that a concrete pavement without transverse joints and containing adequate longitudinal reinforcement is not nearly so susceptible to pumping as pavements of other designs.

In spite of the many transverse cracks that have developed in the long sections, the riding quality of the pavement has remained excellent and the pavement itself has been protected from damaging impact forces such as tend to develop where the surface alinement is not maintained.

SINCE 1938 the Bureau of Public Roads and the State Highway Commission of Indiana have cooperated in making detailed observations of an experimental concrete pavement containing a wide range of continuously reinforced sections. Three published reports[1] have described the scope of the study, the construction of the project, and the observed behavior of the pavement during the first 5 years of service. The present report, which may be considered the major report of the investigation, describes the performance of the various sections over a period of 10 years. Although previously published information is avoided as much as possible, certain essential data are repeated here for both clarity and completeness of the report.

The experimental pavement is a 9–7–9-inch thickened-edge type, 20 feet wide and approximately 6 miles long. It is located near Stilesville, about 30 miles west of Indianapolis, and was constructed during September and October of 1938 as part of the eastbound lanes of the divided highway U S 40.

Traffic counts indicated an average annual daily volume (in both directions) of 3,500 vehicles in 1941 when the pavement was 3 years old. Of these, 1,125 were trucks and busses, the maximum daily gross load being at least 48,000 pounds. In 1948, when the pavement had been in service for 10 years, the average annual daily traffic volume had increased to 5,100 vehicles, trucks and busses comprising 1,280 of the total. At this time the maximum daily gross and axle loads were at least 51,000 and 20,400 pounds, respectively.

Briefly, the experimental pavement consists of sections ranging in length from 20 to 1,310 feet. Incorporated in these sections are various amounts of steel for each of three types of reinforcement. The number and range in length of the individual sections, together with pertinent data on the reinforcing steel used in each, are given in table 1 (p. 4). The lengths of the sections necessary to develop the steel stresses shown in the table were calculated on the basis of certain assumptions as to the resistance offered by the subgrade as the pavement expands and contracts. It was assumed that the resistance would be constant and could be expressed as a coefficient equal to 1½ times the weight of the pavement.

The maximum steel stresses were intended to be such that the elastic limit of the reinforcement would be approached in the longest section of each group, producing, under repeated stressing, inelastic elongation with a consequent opening of the cracks.

Since a wide range in slab end movements was expected in the sections of various lengths, several different widths of transverse joint opening were provided. The shorter sections were separated by conventional dowel-type joints having widths of either ¾ or 1 inch. A joint of a type similar to that frequently used at bridge approaches, and designed to permit a 1½-inch movement in each direction, was placed between intermediate-length sections; whereas for the longer sections provision was made for approximately twice this amount of movement by means of a pair of the bridge-type joints spaced 10 feet apart.

In addition to the regular sections of the experimental pavement—that is, the sections containing continuously bonded steel—four special sections each 500 feet long were included. In these, weakened-plane joints were spaced at 10-foot intervals and the bond between the longitudinal steel and the concrete was broken purposely for a distance of 18 inches on each side of each transverse joint.

---

[1] *Experiments with continuous reinforcement in concrete pavements*, by E. C. Sutherland and S. W. Benham; PUBLIC ROADS, vol. 20, No. 11, Jan. 1940; *Progress in experiments with continuous reinforcement in concrete pavements*, by H. D. Cashell and S. W. Benham; PUBLIC ROADS, vol. 22, No. 3, May 1941; and *Experiments with continuous reinforcement in concrete pavement—A five-year history*, by H. D. Cashell and S. W. Benham; Proceedings of the Highway Research Board, vol. 23, 1943 (condensed).

CONCLUSIONS

In this report the performance of the continuously reinforced sections is traced through the first 10 years of pavement life. The following statements give what appear to be the most significant conclusions to be drawn from the results of this investigation.

Changes in Elevation and Length

1. Changes in pavement elevation were generally small and nonuniform, the lack of uniformity becoming progressively more pronounced, especially during the first 5 years of service. The effect of these nonuniform elevation changes was not apparent in either the length changes or the crack patterns of the sections; but, as would be expected, was reflected in the riding quality of the pavement.

2. Because of the wide range in section lengths an opportunity was afforded to study the effect of subgrade resistance as related to slab movement. The most important conclusions are: (*a*) excepting the very short sections, the daily and annual changes in section lengths are not directly proportional to length of section; (*b*) the magnitude of the restraint offered by the subgrade is a function of the time during which a given temperature or moisture change in the pavement takes place; (*c*) for subgrade soil of the type on which the experimental pavement was constructed, it is estimated that, during the relatively rapid daily length change, the central region of sections greater than approximately 800 feet will be in a state of complete restraint; whereas for the slowly developed annual length change, the central region of sections somewhat greater in length than the longest section (1,310 feet) of this investigation will be completely restrained; and (*d*) for sections of lengths included in this investigation, the data suggest that tensile stresses induced by subgrade resistance are probably larger during the fall than at any other period during the year.

3. Length changes of a progressive or permanent nature developed in sections of all lengths. In the short sections containing comparatively few cracks, it appeared that repeated cycles of moisture and temperature were primarily responsible for such changes. In the longer sections, the tendency of the transverse cracks to open progressively a very small amount was an additional factor contributing to permanent increases.

Development of Cracks

4. Transverse cracks in the experimental sections formed essentially at right angles to the axis of the pavement. The surface widths of these cracks because of slight raveling became, in time, much greater than their real widths. For a given computed maximum steel stress both the surface widths and the real widths of the cracks increased approximately directly with a decrease in the percentage of longitudinal reinforcement. In the heavily traveled lane, after 10 years of service, the average values of the real width of the cracks (obtained in the fall of the year in the central region of the longest section for each percentage of reinforcing steel) ranged from 0.004 inch for the section with 1.82 percent steel to 0.011 inch for the section with 0.45 percent steel. Likewise, the average surface widths of the same cracks ranged from 0.05 to 0.10 inch.

5. The rate of crack development was most pronounced during the early life of the pavement, the greatest rate being between spring and fall of the first year after construction. On the basis of all transverse cracks that developed during the 10-year service period, 67 percent had appeared by the end of the first year. Very few cracks formed during the winter months, indicating that nonuniform changes in pavement elevation caused by frost penetration had little influence upon cracking; and, also, that tensile stresses originating from subgrade resistance were no greater during winter periods than during the fall periods.

6. The study of crack development indicated that the average interval between transverse cracks (average slab length) increased with an increase in section length until a peak value was reached, beyond which there was a rapid decrease in average slab length that became more gradual and finally approached a constant value for the longer sections. If one were interested only in the minimum number of transverse cracks and joints, the data suggest that, in reinforced concrete pavements, the joints should be spaced at approximately 100-foot intervals. It must be kept in mind, however, that this investigation most conclusively shows that the character and not the number of cracks is of the greater importance. In the longer and consequently more heavily reinforced sections, many fine cracks have developed at frequent intervals but these sections have continued to be strong, durable structural units after 10 years of heavy traffic service.

7. The frequency of cracking increased from a minimum value at the end of a section to a maximum value in the central area. For some distance, beginning at the end of the longer sections, the frequency of cracking increased directly with increase in distance. The maximum values of crack frequency, as found in the central area of the sections, increased progressively with increase in section length. It seems reasonable to assume that such values would continue to increase until the sections are long enough to develop complete restraint to slab movement. The 10-year data suggest that, for the conditions obtaining in this investigation, the crack interval in the region of complete restraint might be expected to be approximately 2.0 to 2.5 feet provided, of course, that the reinforcement was adequate.

8. Repetition of traffic loads had only a slight influence on the development of transverse cracks. At the end of 10 years 53 percent of the transverse cracks present in both lanes of the pavement formed in the heavily traveled right-hand lane. However, the greater volume of traffic using the right-hand lane produced more raveling and other superficial damage to the edges of the cracks than the lighter traffic on the left or passing lane, the average surface width of cracks in the heavily traveled lane being approximately three times that of the cracks in the passing lane. Traffic had some effect, also, on the real widths of the cracks, this being less pronounced than in the case of the surface widths.

Effect of Reinforcement

9. All sections were so conservatively designed that the limiting length of section for each percentage of longitudinal steel was not determined.

10. Longitudinal steel reinforcement, within the range of the computed maximum stress values of this investigation, held closed all cracks excepting those in the sections reinforced with the 32-pound wire fabric. In several cases the steel crossing the cracks that formed in the sections containing this light fabric broke, probably from shearing forces. It is indicated that wire fabric as light as 32 pounds per 100 square feet should be used with caution as reinforcement in concrete pavements.

Installing one-fourth inch diameter longitudinal bar reinforcing steel.

11. The presence of the heavy longitudinal bar reinforcement was not in any way detrimental to the condition of the concrete in the pavement, as attested by the complete absence of longitudinal cracking above such bars and by the continued durability of the concrete. In fact, the manner in which the steel held closed all cracks, especially those in the heavily reinforced sections, is believed to be conducive to distributed interfacial pressure and should minimize damage of the concrete from concentrated pressure such as sometimes develops at cracks in plain concrete pavements.

12. The type of reinforcement had only a slight effect on the observed length changes of the sections or on the frequency of cracking in the central portion of long sections; but, for some unknown reason, the reinforcement type seems to have had considerable influence on the average interval between cracks in sections of 300 feet or less in length. On the other hand, the working stresses within the range of the computed maximum values exercised no significant control over the length changes and crack patterns of the sections, probably because of the conservative design assumptions.

13. Heavy reinforcement caused the length changes and crack patterns of the sections to be quite symmetrical about the center of each section, thus indicating a structure of predictable behavior.

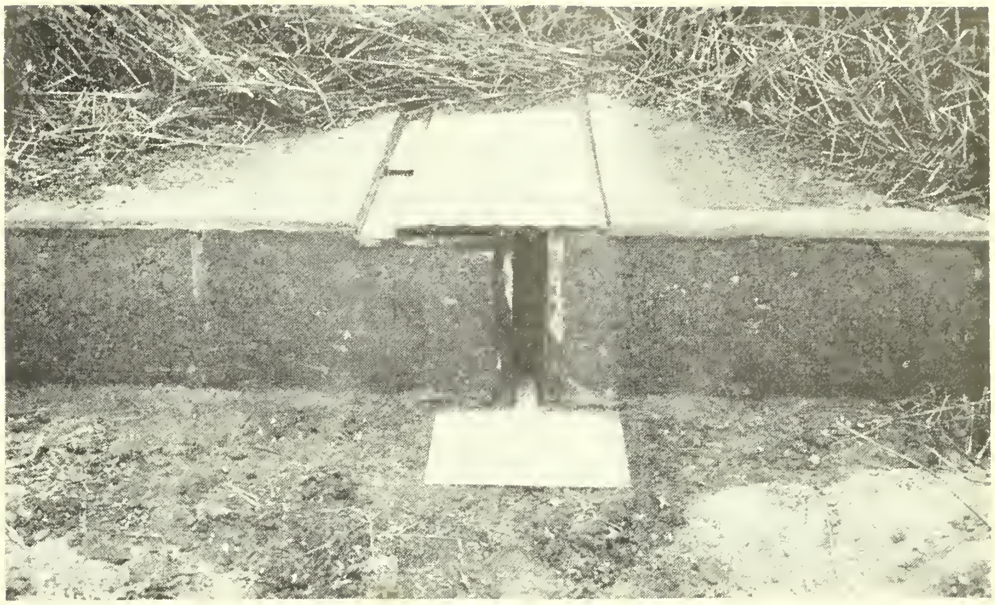
The Special Sections

14. In the four special 500-foot sections, containing warping joints at 10-foot intervals, certain inherent weaknesses developed during the 10 years of traffic service. These weaknesses point out, first, the necessity of providing the warping joints with load-transfer or shear units; and, second, the need of a heavier wire fabric than 45 pounds per 100 square feet. In the halves of the sections provided with shear bars and containing the 91-pound fabric, the steel did not break or inelastically elongate and the pavement remained structurally intact, the riding quality of these halves being nearly the same as that of sections reinforced with continuously bonded steel.

15. During periods of contraction, the continuous reinforcement in the 500-foot special sections exercised considerable control over the length changes of the section as a whole. In spite of the continuity of the reinforcement, however, the warping joints opened progressively with time. This behavior is definitely undesirable since a residual opening of the joints would dissipate all or part of the elastic elongation of the 36 inches of unbonded steel and, in time, may cause failure of the relatively lightweight reinforcement. A corrective measure for the preceding condition would be to decrease the amount of available expansion space.

Pumping and Pavement Smoothness

16. Pumping developed at many expansion joints, but, with two exceptions, was completely absent at the vast number of transverse cracks. This is evidence of the effectiveness of the reinforcing steel in holding tightly together the segments of the sections, thus reducing slab deflections and minimizing the passage of free water to the subgrade soil. The absence of pumping at the submerged-type warping joints of the special sections indicated that the copper seals which enveloped the bottom parting strips prevented the leakage of free water to the subgrade soil.



The bridge-type joint.

17. Relative roughness determinations of the regular sections showed that their surfaces were very smooth initially, and at present are no rougher than some concrete pavements as constructed. The many fine cracks that formed in the long sections have not affected the riding quality of the pavement. The sections containing warping joints at 10-foot intervals were as smooth initially as the regular sections indicating that, with proper care during installation and finishing, closely-spaced warping joints need not affect the initial riding quality of concrete pavements. However, where certain weaknesses have developed, such as faulting of the joints, these special sections have become much rougher than the regular sections.

Economic Benefits

The performance of the Indiana experimental sections has indicated certain economic benefits to be derived from long, continuously reinforced pavement of the type included in this investigation, namely: (1) the fine cracks, even though frequent, ravel only slightly with traffic and exposure, a condition that may be considered superficial and one that will require no maintenance; (2) except in localized areas of extremely poor subgrade, pumping will not develop, thus minimizing the need for base courses or other expensive subgrade treatments; and (3) the riding quality of the pavement might be expected to remain excellent and the pavement itself would be protected from damaging impact forces such as frequently develop at faulted joints and cracks.

More recently, other researches relating to the use of continuously bonded longitudinal reinforcement have been inaugurated.[2] The experimental sections in these pavements are all longer than the longest section of the Stilesville experimental pavement and are less heavily reinforced. Sections of various thicknesses, reinforced with various percentages of longitudinal steel, and constructed both with and without subbases, are included. While the pavements have not been in service long enough to permit conclusions to be drawn, it seems probable that eventually they will provide considerable additional data on the relative economies of continuously reinforced pavements and those which are designed as a series of comparatively short, independent slabs.

Thus, it is possible that, when all factors are considered, a concrete pavement without joints and reinforced with continuous bonded steel in sufficient amount to resist all stresses safely and to hold all cracks closed may, in many cases, cost no more than current designs of concrete pavement which include greater slab thickness, lighter reinforcement, transverse joints, and subgrade treatment.

SCOPE OF DISCUSSION

The discussion of the 10-year performance of the pavement is presented in six parts, as follows: (1) Periodic elevation changes of the regular sections; (2) daily, annual, and progressive changes in the length of the regular sections; (3) development, distribution, and present condition of cracks in the regular sections; (4) behavior of the 500-foot special sections; (5) occurrence of pumping; and (6) smoothness of the pavement. Pertinent data on the pavement sections and the reinforcing steel in them are given in table 1.

---

[2] *Continuously reinforced concrete pavements without joints*, by W. R. Woolley; *Preliminary report on current experiment with continuous reinforcement in New Jersey*, by W. Van Breemen; and *An experimental continuously reinforced concrete pavement in Illinois*, by H. W. Russell and J. D. Lindsay; Proceedings of the Highway Research Board, vol. 27, 1947.

Table 1.—*Details of reinforcement in the experimental pavement*

Number of sections for each length [1]	Length of each section [2]	Calculated maximum stress in steel	Weight of reinforcement	Reinforcement size and spacing				Percentage of longitudinal steel [4]	Average tensile strength of longitudinal steel	
				Longitudinal		Transverse			Yield point	Ultimate
				Diameter [3]	Spacing, c. to c.	Diameter [3]	Spacing, c. to c.			
	Feet	*Lb. per sq. in.*	*Lb. per 100 sq. ft.*		*Inches*		*Inches*	*Percent*	*Lb. per sq. in.*	*Lb. per sq. in.*
RAIL-STEEL BARS (DEFORMED)										
2	600	25, 000		1-inch	6	½-inch	24	1. 82	63, 300	113, 200
	840	35, 000								
	1, 080	45, 000								
	1, 320	55, 000								
4	340	25, 000		¾-inch	6	½-inch	24	1. 02	64, 400	113, 300
	470	35, 000								
	610	45, 000								
	740	55, 000								
4	150	25, 000		½-inch	6	½-inch	24	. 45	68, 800	115, 300
	210	35, 000								
	270	45, 000								
	330	55, 000								
6	80	25, 000		⅜-inch	6	⅜-inch	24	. 26	66, 700	93, 600
	120	35, 000								
	150	45, 000								
	180	55, 000								
6	40	25, 000		¼-inch	6	¼-inch	12	. 11	60, 300	84, 600
	50	35, 000								
	60	45, 000								
	80	55, 000								
BILLET-STEEL BARS (DEFORMED), INTERMEDIATE GRADE										
2	360	15, 000		1-inch	6	½-inch	24	1. 82	46, 900	78, 000
	600	25, 000								
	840	35, 000								
	1, 080	45, 000								
4	200	15, 000		¾-inch	6	½-inch	24	1. 02	49, 100	78, 500
	340	25, 000								
	470	35, 000								
	610	45, 000								
4	90	15, 000		½-inch	6	½-inch	24	. 45	51, 400	78, 600
	150	25, 000								
	210	35, 000								
	270	45, 000								
6	50	15, 000		⅜-inch	6	⅜-inch	24	. 26	55, 500	81, 900
	80	25, 000								
	120	35, 000								
	150	45, 000								
6	20	15, 000		¼-inch	6	¼-inch	12	. 11	56, 900	77, 300
	40	25, 000								
	50	35, 000								
	60	45, 000								
WIRE FABRIC (COLD-DRAWN WIRES)										
6	140	25, 000	149	No. 0000	4	No. 3	12	. 42	------	81, 800
	190	35, 000								
	250	45, 000								
	310	55, 000								
6	90	25, 000	107	No. 0000	6	No. 3	12	. 28	------	80, 300
	130	35, 000								
	170	45, 000								
	200	55, 000								
6	80	25, 000	91	No. 000	6	No. 4	12	. 24	------	89, 100
	110	35, 000								
	140	45, 000								
	170	55, 000								
6	60	25, 000	65	No. 0	6	No. 6	12	. 17	------	83, 700
	80	35, 000								
	100	45, 000								
	120	55, 000								
6	30	25, 000	45	No. 3	6	No. 6	12	. 11	------	81, 000
	50	35, 000								
	60	45, 000								
	80	55, 000								
6	20	25, 000	32	No. 6	6	No. 6	12	. 07	------	88, 700
	30	35, 000								
	40	45, 000								
	50	55, 000								

[1] The term "section" as used in this report refers to a lane or 10-foot width of pavement; thus the number "2" indicates a pair of sections, one being on each side of the center joint.

[2] The lengths of the longer sections are nominal lengths and may be either 5 or 10 feet greater than the actual length in cases where a pair of bridge-type joints were installed.

[3] The rail-steel and billet-steel reinforcement were round bars. The diameters of the wires in the fabric were as follows: No. 0000, 0.3938 inch; No. 000, 0.3625 inch; No. 0, 0.3065 inch; No 3, 0.2437 inch; No. 4, 0.2253 inch; No. 6, 0.1920 inch.

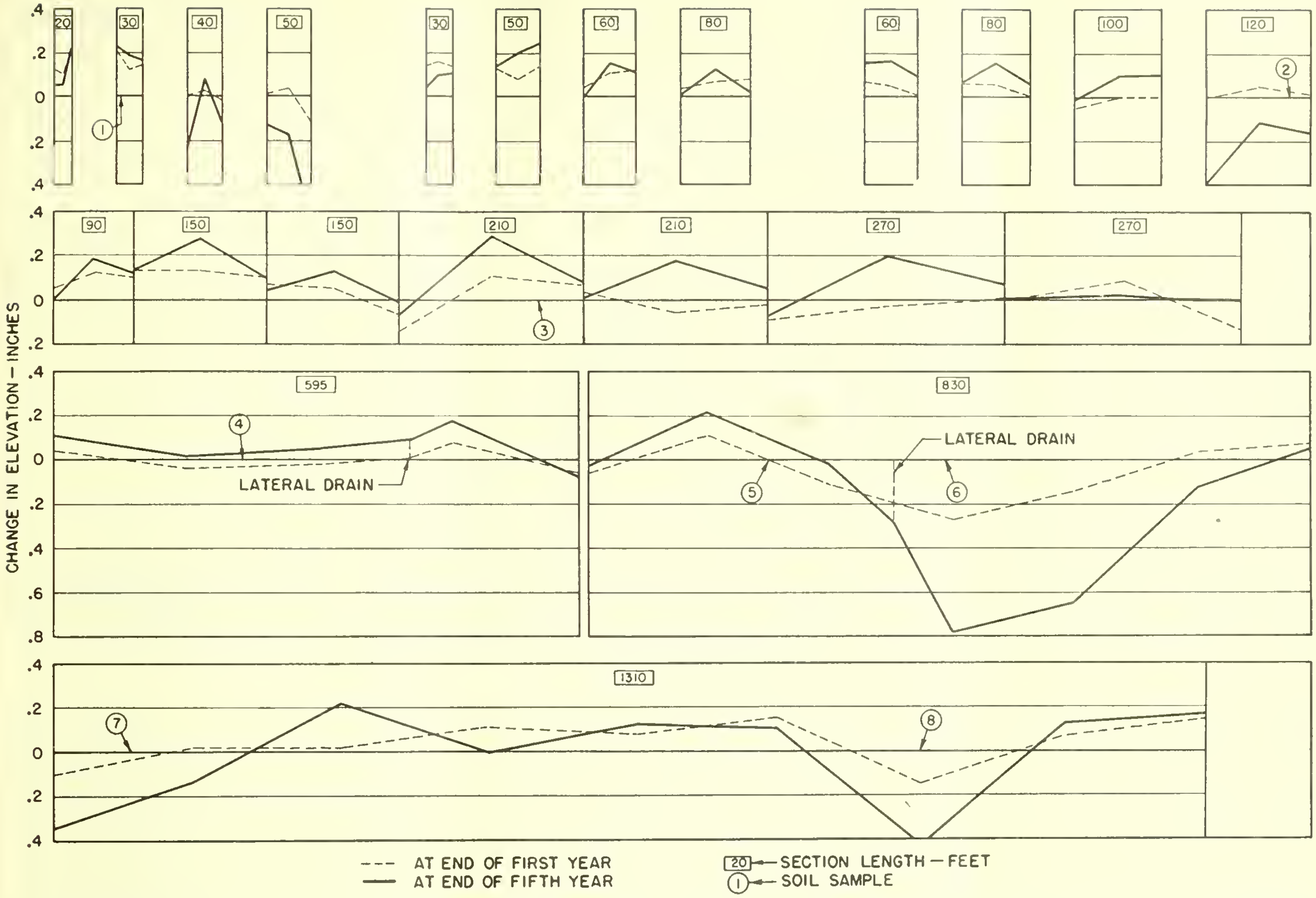
[4] Cross-sectional area of the longitudinal steel expressed as a percentage of the cross-sectional area of the concrete slab.

Part 1.—PERIODIC ELEVATION CHANGES

Three sets of precise elevation measurements have been made over the entire length of the experimental pavement: In the late fall of 1938 or shortly after construction of the sections; in the fall of 1939 or approximately 1 year after construction; and in the severe winter of 1939–40 when the frost had penetrated the ground to a depth of about 20 inches. In addition, during the first 5 years, elevation measurements were made at more frequent intervals over selected sections. All such measurements were made on reference points installed in the right-hand or heavily traveled lane of the pavement.

At the end of the first year the majority of the elevation changes were very small. Specifically, only 7 percent of the 487 midlane locations at which measurements were made showed a change in elevation greater than one-fourth inch when compared with the base elevations established soon after construction.

During the peak of the severe winter of 1939–40 increases in elevation were generally within the range of 0.2 to 1.0 inch as compared



SOIL SAMPLE NO.	1	2	3	4	5	6	7	8
SILT – PERCENT	38	43	60	58	63	39	46	43
CLAY – PERCENT	16	18	18	18	13	12	19	18
LIQUID LIMIT	32	34	36	37	43	28	35	29
PLASTICITY INDEX	14	11	12	12	19	11	14	11

SOIL SAMPLE NO.	1	2	3	4	5	6	7	8
	MOISTURE CONTENT – PERCENT							
0 – 3 INCH DEPTH	16.0	11.0	13.6	11.5	12.9	19.2	16.8	9.8
3 – 12 INCH DEPTH	18.0	13.0	18.0	14.6	15.8	13.5	18.7	15.7
12 – 24 INCH DEPTH	18.0	19.0	18.0	13.1	20.9	13.5	19.6	12.8

Figure 1.—Changes in elevation of selected sections at the end of the first and fifth years of pavement life, and physical characteristics of the subgrade soil at the time the pavement was placed.

with the elevations determined the previous fall. It was observed also, in most instances, that heaving was greater at the expansion joints than at points elsewhere in the sections, averaging 0.47 inch at 151 expansion joints and 0.33 inch at 185 points elsewhere in the sections. These data emphasize the importance of tightly sealed joints in pavements exposed to freezing conditions.

Examples of typical changes in pavement elevation are shown in figures 1 and 2.

The elevation changes in figure 1 are those observed on selected sections in the fall at the end of the first and fifth years of pavement life, using as a base the elevations established shortly after construction. Subgrade soil data applicable at the time the pavement was placed are also given in this figure.

The data of figure 1 indicate that the elevation changes at the end of 5 years were appreciably greater in magnitude and were less uniform than at the end of the first year. The greatest change was a settlement of 0.8 inch near the center of the 830-foot section. However, measurements taken in this area at the end of 10 years showed virtually no change with respect to the 5-year profile.

Figure 2 shows the changes in elevation of a composite section, this figure being prepared from measurements made from time to time at the centers and at the ends of 24 representative sections, using as a base the elevations established soon after construction. Thus, the elevation changes shown for the center and for the end of the composite section are, respectively, the average changes in elevation of the 24 centers and of the 48 ends of the representative sections.

The positions of the center and end of the composite section indicate that the pavement as a whole raised slightly with respect to the basic elevation; and that, excepting in the severe winter of 1939–40, the changes in elevation were greater at the center than at the end, suggesting a permanent downward warping at the end of the composite section with respect to its center. It is believed that soil displacement resulting from pumping or consolidation of the subgrade at the joints may be primarily responsible for this condition of distortion.

Part 2.—CHANGES IN LENGTH

Daily, annual, and progressive changes in length were measured at the ends of a number of representative sections. These length changes were carefully determined either by measurement to fixed reference points located at the ends of a section or by measurements across joints between sections of equal length. Cross-joint measurements give the width changes of joints which, when determined for a joint separating sections of equal length, should approximate the total length change of one of the joining sections. All of these measurements were from points installed in

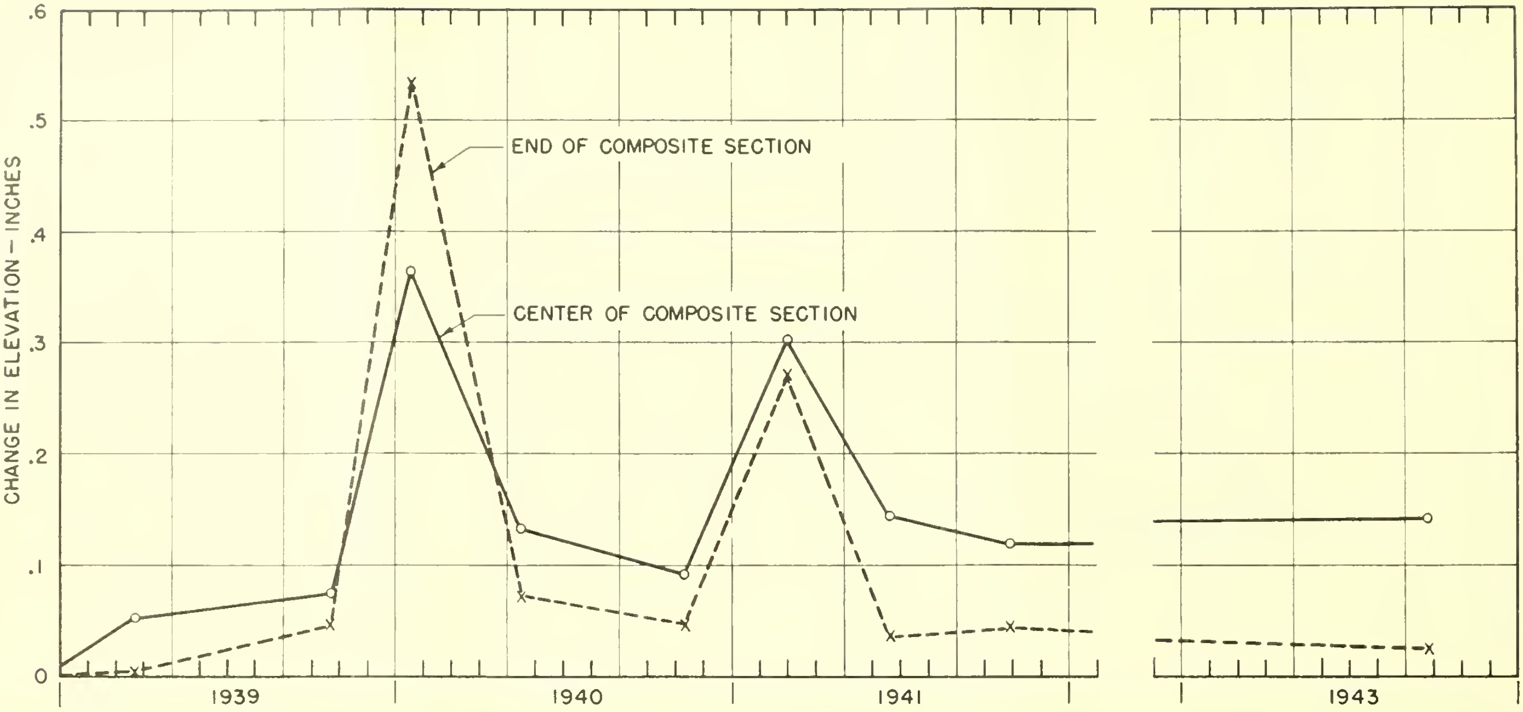


Figure 2.—Changes in elevation of the center and end of a composite section (average of 24 representative sections); base measurements obtained in December 1938.

the surface of the right-hand or heavily traveled lane of the pavement.

It will be recalled that all transverse joints were designed to care for a reasonable amount of slab expansion and, as far as can be determined, the observed length changes of the sections were unaffected by restraint at the joints during the first 5 years of pavement life. Considerable care was exercised during construction in correctly alining the round steel dowels used in the joints which separate the shorter sections, so that restraint from this source was reduced to a minimum. Over a period of time, however, the joints, especially the bridge-type joints, gradually became filled with soil and with bituminous material used for their maintenance so, at present, the movements of several of the longer sections may be restrained to some extent during periods of maximum expansion.

For a given temperature or moisture change in the concrete, there should be a proportionate change in section length provided the section remained structurally intact and was not restrained in any manner. Actually, however, the length changes of the sections of this study are affected by any restraint that may develop at transverse joints and by such factors as subgrade resistance to slab movement, differences in the thermal coefficients of steel and concrete, moisture changes of the concrete and not of the steel, and changes in width of existing transverse cracks. Just how much influence each of these factors exerts cannot be determined, but it is believed that the subgrade resistance is the most important. Since the thermal coefficients of steel and concrete are nearly the same (that for steel being somewhat the greater) it seems reasonable to expect that this factor would have little effect on the length changes of the sections, particularly during expansion. As for transverse cracking, it will be shown later in the report that very few cracks occurred in sections having lengths less than 120 feet and the cracks that developed in the longer, more heavily reinforced sections were extremely fine. Thus, for either a daily or annual time period, the cumulative change in width of all transverse cracks in a given section should be small compared to the overall length change of the section.

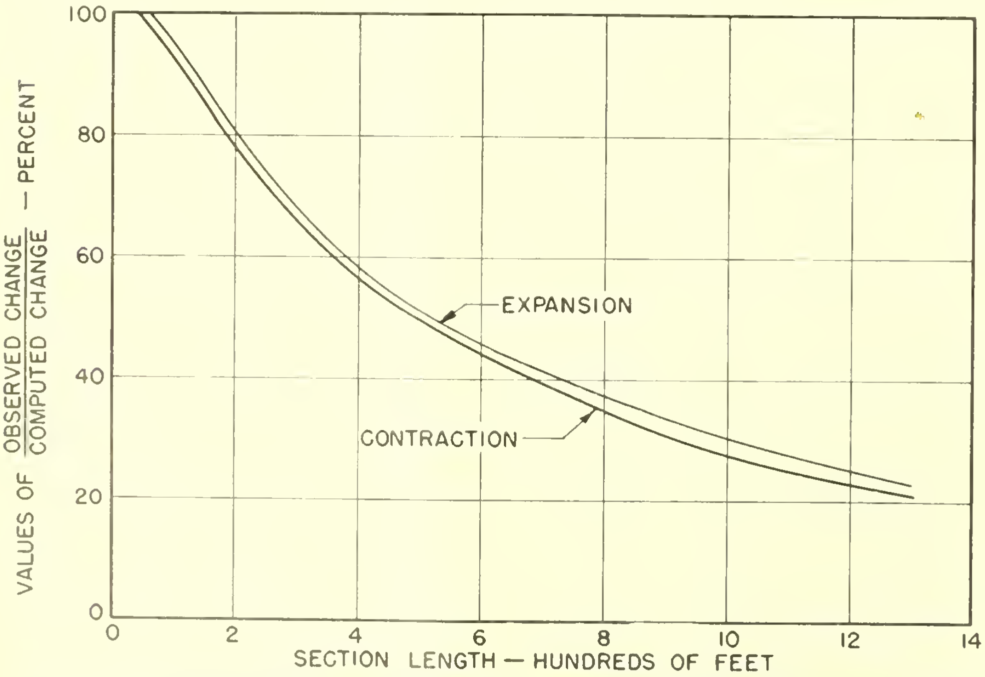


Figure 3.—Observed daily changes in section lengths expressed as percentages of the computed changes in length of equivalent unrestrained sections.

Daily Length Changes

In considering the daily length changes it will be recalled that all measurements at the joints were made at the level of the upper surface of the pavement and were, therefore, affected in some degree by changes in the condition of warping at the ends of the slabs. In this investigation it was found that the corrections for daily warping ranged from 0.003 to 0.005 inch for days when a large temperature change occurred in the pavement. These corrections were applied only to sections having lengths of 60 feet or less because in the longer sections the magnitude of the length changes was such as to make the correction unimportant.

In figure 4 are shown the relations between section length and change in section length as found for a daily mid-depth pavement temperature drop of 24° F. and a daily mid-depth pavement temperature rise of 30° F. The data for this figure were obtained from 64 sections that cover the range of section lengths for all percentages of longitudinal steel included in each of the three types of reinforcement. The slopes of the straight lines shown in the figure were computed from the change in length of 19 uncracked sections, 20 to 60 feet long, and thus represent, for the temperature changes mentioned, the rates of length change for short sections that are comparatively free to expand and contract. From these the coefficients of daily length change for short sections, which should approximate the thermal coefficient of the concrete, are readily obtained.

It is apparent from this figure that sections up to approximately 75 feet in length move with as much freedom as the very short sections. The change in length of sections greater than about 75 feet is restrained by subgrade resistance and perhaps other factors and this restraint, the effect of which is shown as departure from the slope line established by the short sections, increases rapidly as the sections become longer. After a section length of about 800 feet is reached, the curves level off, indicating that the maximum restraint has been developed and that sections whose lengths are greater than this may be expected to show total length changes of equal magnitude. This suggests that the central portion of sections greater than 800 feet will be completely restrained during quick changes in average concrete temperature. Applied to the 1,310-foot section, this would mean that the central 500± feet of this section did not move during the daily temperature changes for which data are shown in figure 4.

The great influence of subgrade resistance on the relatively rapid daily change in length of the sections is clearly indicated in figure 3. This figure, prepared from the curves of figure 4, shows the observed change in length of a section expressed as a percentage of the change in length of an unrestrained section of the same length. The length changes of the unrestrained sections were calculated from the data obtained from the very short sections. Reduced to this basis, the observed daily length changes were about the same for both the expansion and contraction cycles.

In the discussion of figure 4 it was suggested that the central portion of long sections may be completely restrained during quick changes in pavement temperature. This seems to be confirmed by the data given in figure 5, data which show the longitudinal movements observed at the center, quarter-points, and ends of a 1,310-foot section for a daily mid-depth temperature drop of 19° F. and a rise of 25° F. The values shown for the quarter-point and for the end are in each case the average value obtained from measurements at both quarter-points and at both ends of the section.

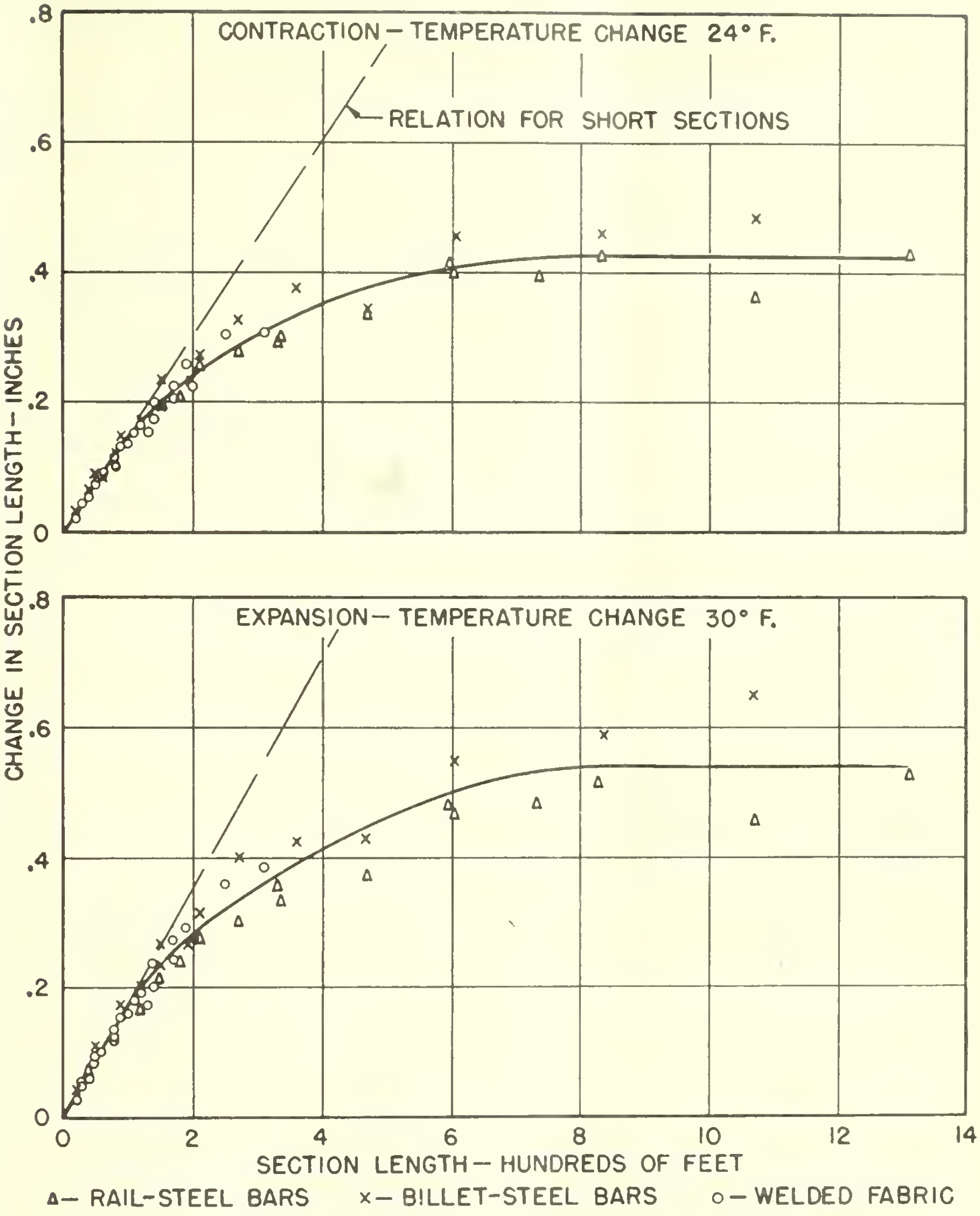


Figure 4.—Relation between section length and daily change in length.

These data indicate that during contraction the movement at the ends of the 1,310-foot section was about 20 percent and that at the quarter-points about 1 percent of the movement which would be found in an unrestrained section of the same length. Corresponding values for expansion were about 20 and 2 percent. It will be noted that between 500 and 550 feet of the central portion of the 1,310-foot section did not move during the daily cycle, this length being comparable to the 500±-foot length that was estimated from the results of figure 4.

The manner in which certain sections respond to a daily rise in pavement temperature is shown in figure 6. The basic measurements for this figure were obtained in the early morning of a summer day and subsequent measurements were made at intervals until late afternoon.

In the case of both the 470- and 1,310-foot sections the relation between increase in temperature and change in over-all section length remained linear until a total elongation of approximately 0.2 inch was attained, after which the rate of length change increased progressively with temperature, being more pronounced for the longer section. For example, during the 28° F. temperature rise shown in figure 6, the 1,310-foot section moved 0.19 inch for the first 14° temperature increase and 0.27 inch for the second 14° increase. Since the movements of the sections are intimately related to the restraint offered by the subgrade, the increase in the rate of length change of the long sections suggests that, after a certain amount of slab displacement, the total or accumulated subgrade resistance continues to increase, but at a progressively decreasing rate.

Daily changes in length of a limited number of sections were observed each summer over a 4-year period on days when a large temperature change occurred in the pavement. This 4-year period extended from the second through the fifth year of pavement life and, therefore, it is believed that the movements

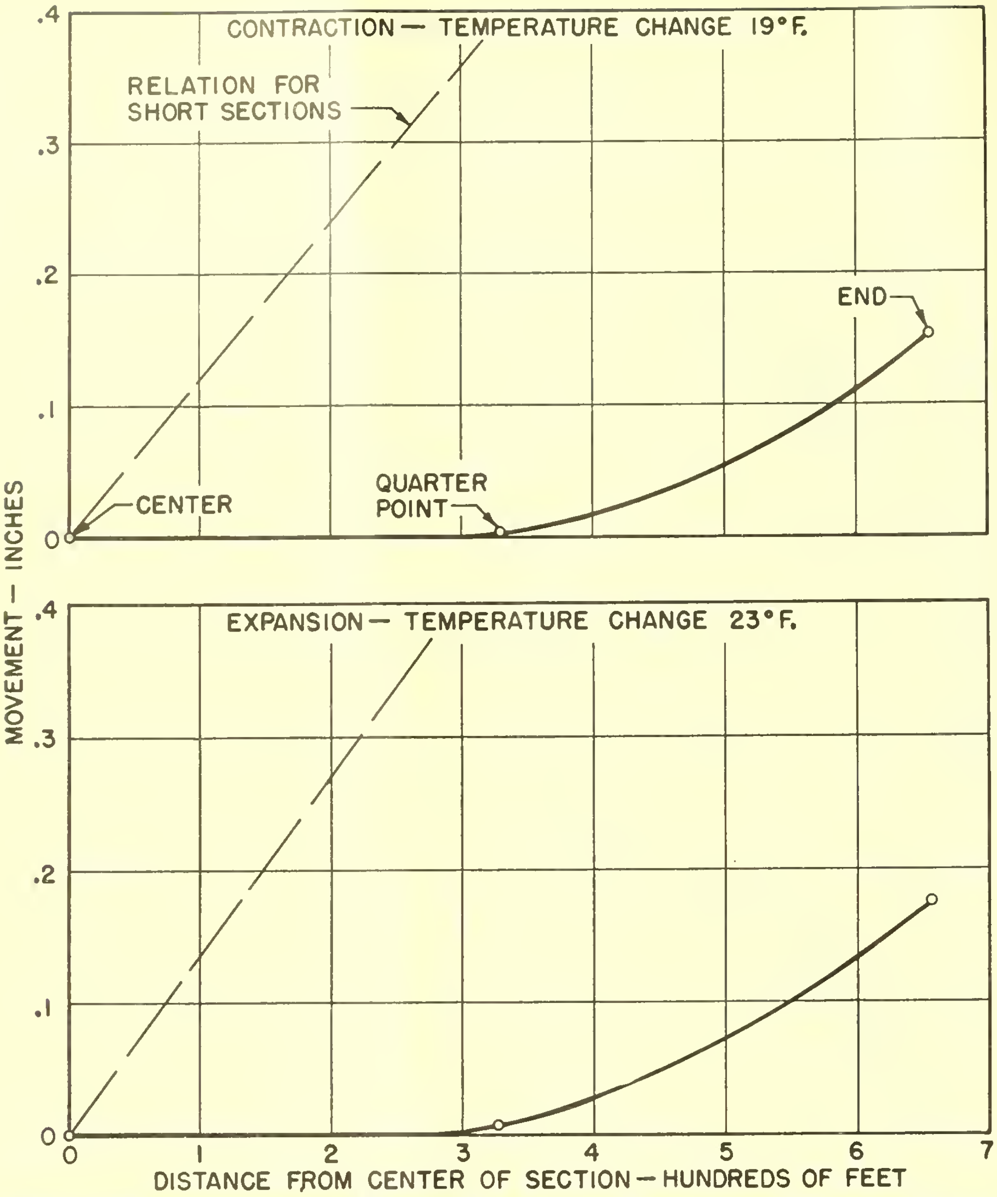


Figure 5.—Daily movement at the center, quarter point, and end of a 1,310-foot section.

of the sections were not affected by restraint at the transverse joints. Table 2 gives the daily length change data for the various section lengths, reduced to unit values per degree F. These values indicate that the coefficient of the daily length change of a given section did not change appreciably from year to year.

Table 2.—*Summary of values of coefficients of daily length change (based on changes in over-all section length)*

Section length	Unit change in section length per degree F.×10⁻⁷					
	July 1940	June 1941		June 1942		July 1943
	A. M. to P. M.	P. M. to A. M.	A. M. to P. M.	P. M. to A. M.	A. M. to P. M.	A. M. to P. M.
Feet						
20	49	49	49	41	52	---
150	---	46	43	44	41	---
335	29	32	29	---	---	---
470	---	25	25	22	24	---
600	21	24	24	21	23	---
1,070	13	13	14	---	---	--
1,310	9	11	11	8	10	10

The average of the five values obtained from the observed daily length change for the 20-foot section (table 2) is 0.0000048 per degree F. Because the 20-foot section is free to expand and contract when subjected to a temperature change, this value should approximate the thermal coefficient of the concrete. As a matter of supporting data the slopes established by the short sections, as shown in figure 4, when divided by the temperature change of the concrete, give coefficient values of 0.0000053 per degree F. for contraction and 0.0000049 per degree F. for expansion.

Annual Length Changes

Figure 7 contains the annual length change data for the various sections for the first, second, third, and fifth years of the life of the pavement. The annual change in length of a section was computed from data obtained in the morning of a midwinter day and in the afternoon of a midsummer day and, consequently, includes the length change that occurred between the morning of a winter day and the morning of a summer day plus the daily length change that occurred between the morning and afternoon of the aforementioned summer day. Since an effort was made to obtain these data during the coldest period of winter and the hottest period of summer, the length changes shown are approximately the maximum for the annual cycle. The slopes of the straight lines represent respective annual relations determined from 19 uncracked short sections.

The type of reinforcement used in the various sections is denoted by symbol. There appears to be some tendency for sections containing billet-steel bars to develop slightly greater annual length changes than equivalent-length sections reinforced with rail-steel bars. This same tendency was noted in the daily expansion and contraction data of figure 4. The cause for this difference is not known.

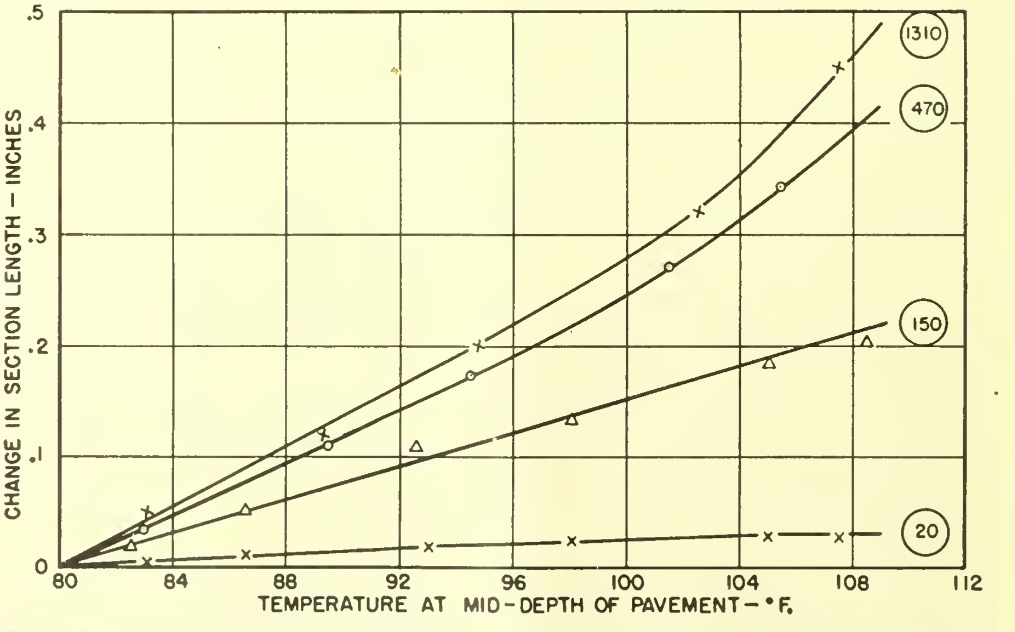


Figure 6.—Effect of a daily rise of the mean pavement temperature on the change in length of several sections (figures in circles indicate the length of sections in feet).

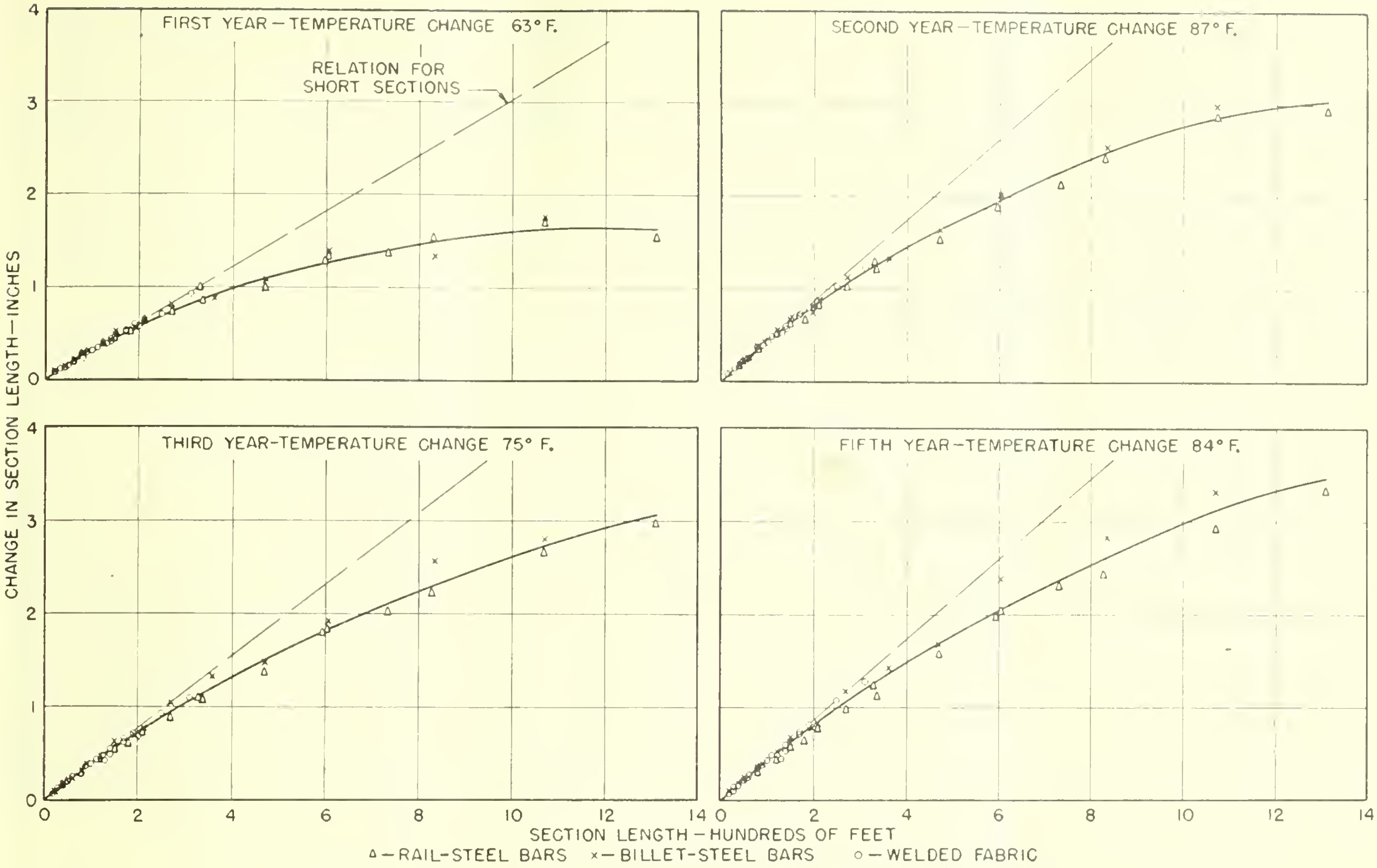


Figure 7.—Relation between section length and annual change in length.

It is indicated by the four curves of figure 7 that sections up to approximately 150 feet in length move with as much freedom during an annual cycle as do the very short sections. The length changes of sections greater than 150 feet are restrained by the subgrade, however, and this restraint increases progressively with increase in section length. The data of figure 4 indicated that daily restraint to free movement was first noticeable in sections about 75 feet long. It should be remembered that the annual length change data considered above include the effects of one daily cycle also. A probable explanation for the observed difference, just mentioned, is that, under the slowly developed temperature rise from winter to summer, sections up to at least 150 feet in length moved freely because they encountered less restraint from the subgrade than obtains during the more rapid daily cycle of length change. Hence, the small amount of daily restraint to free movement of sections between 75 and 150 feet in length, as shown in figure 4, while present is not apparent in the curves of figure 7.

In connection with this study of the annual length changes of the sections it is of interest to note the symmetry of movement that was found in the long sections. For example, during the fifth annual period, the observed movement at one end of a 1,070-foot section was 1.50 inches while at the other end the movement was 1.42 inches; likewise the movements at the two ends of the 1,310-foot section were 1.62 and 1.72 inches, respectively.

To provide a more easily visualized comparison of the annual length changes of the various sections, figure 8 was developed from figure 7 in the same manner that figure 3 was obtained from figure 4. The curves of figure 8 show not only the magnitude of the restraint that was present in the various sections, but also that the sections expanded more freely

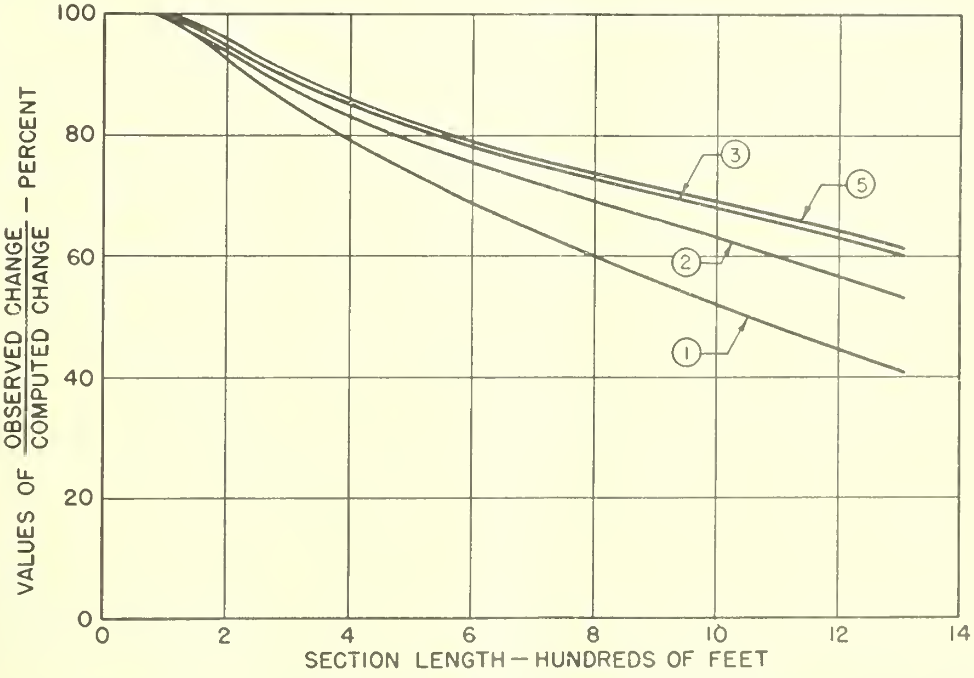


Figure 8.—Observed annual changes in section lengths expressed as percentages of the computed changes in length of equivalent unrestrained sections (figures in circles indicate age of pavement at time of observations).

progressively for each of the first three annual cycles. Thus, it appears that the sections encountered less subgrade resistance with each successive annual expansion period until, by the end of the third period, a condition of essential stability was reached.

Further evidence is added by the annual movements observed at the quarter-points of the 1,310-foot section. For the first year the annual movement at the quarter-points was about 10 percent of the movement to be expected at the quarter-points of an unrestrained section of equal length. For the second, third, and fifth years the values were respectively 31, 46, and 45 percent.

Greater Freedom of Movement Annually Than Daily

Table 3 shows the annual length changes of selected sections reduced to unit values per degree F. These annual coefficients of length change, although expressed as unit values per degree F., involve temperature, moisture, subgrade resistance, and perhaps other factors. The factor of moisture will be discussed later in the report.

In comparing the coefficient values of the longer sections of table 3 with those of table 2, it is observed that coefficients for an annual expansion period are, in general, much greater than those for a daily expansion period, thus suggesting greater freedom of movement of the sections during an annual period.

This condition is clearly shown, also, by comparing the curves of figure 8 with those of figure 3. For example, during the annual length-change cycle for the 1,310-foot section (fifth year) the observed movement was 62 percent of the theoretical length change for an unrestrained section of equal length; whereas during the daily length change the value was only 22 percent. Hence, it is strongly indicated that the magnitude of the restraint offered by the subgrade is a function of the time during which a given temperature or moisture change in the pavement takes place.

Data lending further support to this observation are given in figure 9. This figure shows the length changes of the sections that occurred between the morning of a day in February when the mid-depth pavement temperature was 32° F. and the morning of a day in late June of the same year when the mid-depth pavement temperature was 77° F. Therefore, the data do not include the effect of a quick daily temperature rise, but rather show only the comparative freedom with which sections of all lengths expanded under a slowly developed temperature rise of 45° F. These measurements were made during the third year of the life of the pavement, at which time stablization of annual movement had developed.

Table 3.—*Summary of values of coefficients of annual length change (based on changes in over-all section length)*

Section length	Unit change in section length per degree F.×10⁻⁷			
	First year	Second year	Third year	Fifth year
Feet				
20	40	42	43	43
150	40	40	41	41
335	34	35	36	36
470	29	32	34	34
600	29	31	34	35
1, 070	21	26	28	29
1, 310	15	21	25	25

It appears from the cycle of length change shown in figure 9 that sections up to about 900 feet in length expanded as freely as the very short sections. The free movement of sections greater than 900 feet is restrained and, although the sections included in this study are not long enough to warrant definite conclusions, it is indicated by the rapid increase in restraint that the central portion of sections greater in length than approximately 1,700 to 1,800 feet would be in a state of complete restraint during an annual cycle.

The manner in which the sections up to 900 feet in length expanded from February to June indicates that subgrade resistance did not accumulate in these sections and, as a consequence, residual compression was probably absent on the morning of the June day when the pavement temperature was 77° F. Therefore, it seems logical to expect that as summer advances and the mean pavement temperature gradually rises, sections of considerable length, unless restrained at the joints, expand to their annual maximum without developing appreciable residual compression. If this is the case, then it would be expected that in late summer or early fall the comparatively large, sudden drops in temperature would cause comparatively large, direct tensile stresses to be developed in the sections, larger probably than at any other period during the year.

Again confirmatory evidence is supplied by the movements observed at the quarter-points of the 1,310-foot section. During the fifth year of pavement life, a 0.61-inch movement was recorded at the quarter-points of this section when it had expanded to its approximate maximum length for the annual cycle. During the early fall of the same year, however, after the pavement temperature had dropped approximately 50 percent of its winter to summer rise, the contraction of the section was restrained to the extent that the return movement at its quarter-points was only 0.06 inch or about 10 percent of the movement observed at maximum expansion.

In concluding this discussion of the annual length changes of the sections, it is of interest to compare sections in which the three different types of reinforcement were used and, also, those in which the maximum stresses in the longitudinal steel presumably varied considerably, in order to determine the effect of these factors on the relation between section length and annual contraction of the sections. These comparisons are shown in figure 10 for a contraction period since, during such a period, maximum tensile stresses develop in the reinforcement. The length changes given in this figure are the result of a 77° F. fall in temperature that occurred between midsummer and midwinter of the third annual contraction period.

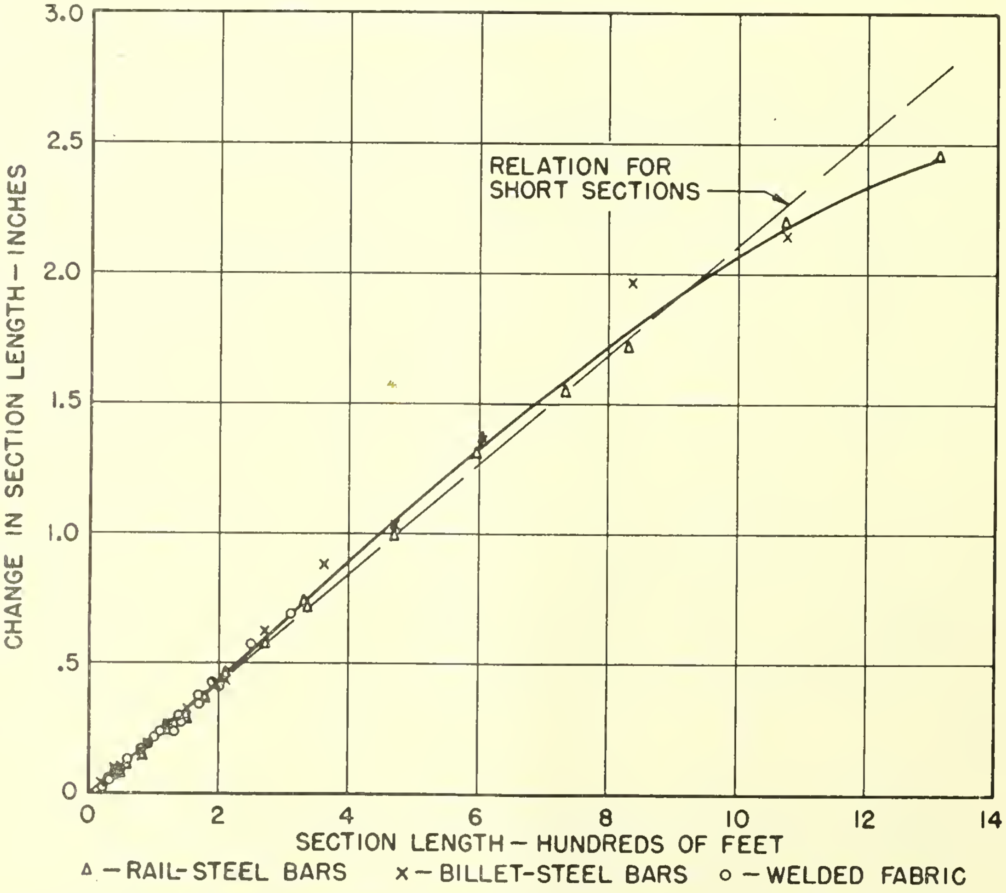


Figure 9.—Relation between section length and change in length from the morning of a winter day to the morning of a summer day of the same year (mean pavement temperature change of 45° F.).

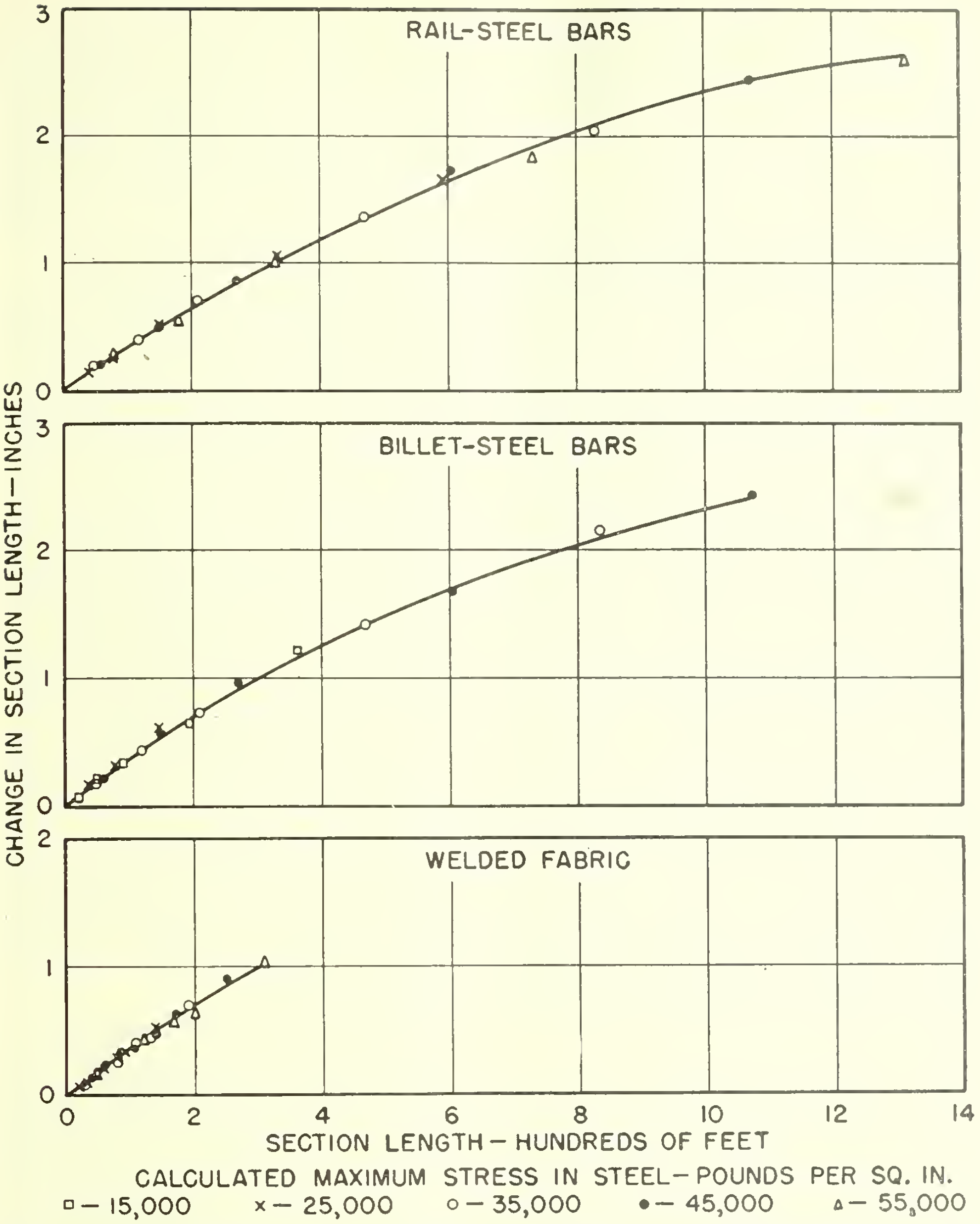


Figure 10.—Effect of type of reinforcement and calculated maximum steel stress on the relation between section length and annual contraction (77° F. temperature drop).

A comparison of the three curves of figure 10 indicates that, for a contraction period, the type of reinforcement had little influence on the observed length changes of the sections. For example, in the case of a 300-foot section the measured length changes for the rail-steel bars, the billet-steel bars, and the welded fabric were 0.92, 0.98, and 1.00 inch respectively.

The maximum steel stresses as calculated for the various sections are denoted, in figure 10, by symbol. For a given section length these stress values may be considered as inverse indices of steel area. The orderly manner in which all points, regardless of symbol, fall on the curves in the figure is evidence that, within the ranges available for comparison in a given section length, the amount of the longitudinal steel exercises no significant control over the length changes. For example, two sections each approximately 600 feet long, reinforced with rail-steel bars, show essentially the same length change although one contains 1.82 percent of longitudinal steel while the other contains but 1.02 percent.

An examination of all of the transverse cracks in the regular sections indicated that, except for 3 in the 24 sections containing the 32-pound wire fabric, all were held closed by the longitudinal steel. The condition of these cracks will be discussed later.

Since after 10 years of heavy-duty service none of the cracks in the regular sections showed evidence of inelastic deformation of the longitudinal steel (except the three just mentioned), it must be concluded that the assumptions used in computing steel stresses in the original design lengths were unduly conservative. How closely the elastic limit of the steel has been approached during this period of service remains unknown.

Progressive Length Changes

To evaluate length changes of a progressive or permanent nature, measurements were made at the ends of a number of selected sections every February and August during the first 9 years of pavement life. The February observations were obtained when the mid-depth slab temperature was approximately 32° F. and the August observations when the mid-depth slab temperature was approximately 92° F. The effect of moisture on the determination of the progressive length changes of the sections was minimized, as much as possible, since there is reason to believe that the moisture content of the pavement remains virtually stable in February and in August. Studies indicate that during these months the absorbed moisture is at the maximum and minimum, respectively, of the annual cycle of moisture change.

The data from the study of progressive or permanent changes in the lengths of the sections are given in figures 11 and 12.

In figure 11 the broken lines show the average February to August and August to February unit length changes of a number of short, uncracked sections, plotted with respect to the initial set of measurements obtained in February of 1939. These sections are 20 to 80 feet in length, comprise 340 feet of pavement in total, and are relatively free to expand and contract. The length changes are expressed as unit values per 60° F. change in slab temperature.

The solid line drawn through the mean

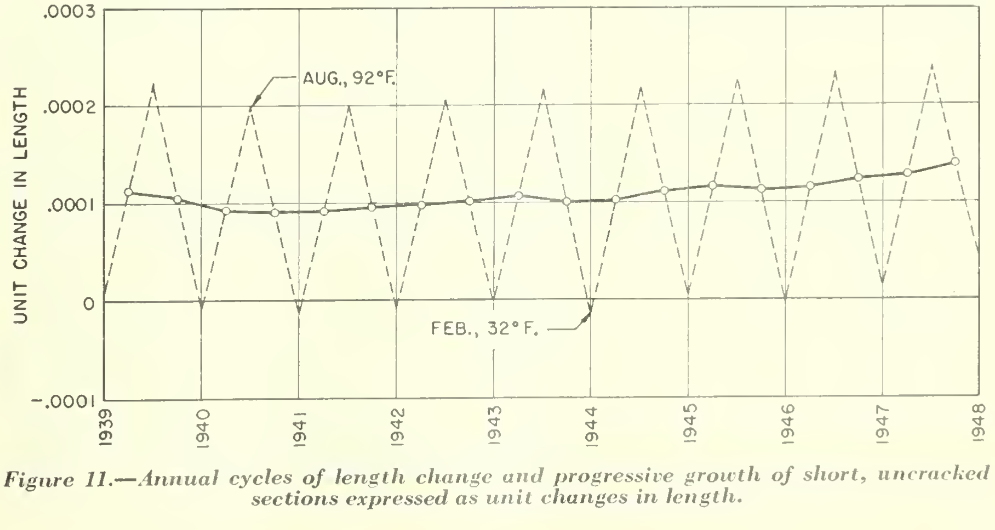


Figure 11.—Annual cycles of length change and progressive growth of short, uncracked sections expressed as unit changes in length.

points of the cyclic variations of figure 11 indicates a progressive permanent growth or increase in the length of the sections, this growth being more pronounced after the fifth year of pavement life. In fact, the growth was so small during the early life of the pavement that in the 5-year report it was stated that "no definite indication of a permanent change in the length of the short sections was observed". Since these short sections are structurally intact there can be little doubt that a permanent increase in length is developing. This appears to be another instance of the tendency of some concretes, at least, to grow when subjected to repeated cycles of temperature and moisture change.

The difference between the high and low points of the mean line of figure 11 represents a permanent unit increase of 0.000048 or an increase of approximately one-sixteenth inch for a 100-foot slab. A similar study of permanent growth of concrete pavement was made at the Arlington Experiment Farm, Virginia, by the Bureau of Public Roads [3] on a 40-foot, plain concrete test section. Over a 9-year period this test section showed a permanent increase in length equal to approximately three-eighths inch for a 100-foot slab. This value is approximately six times greater than the value computed from the data of the reinforced short sections of the Indiana pavement. Whether the reinforcing steel in the Indiana sections restrained the tendency for the concrete to grow in the presence of moisture or whether differences in material and exposure between the sections of the two investigations were responsible for this difference in behavior can only be a matter of speculation.

Effect of Moisture

Although the data in figure 11 were obtained primarily for a study of length changes of a permanent nature, values of winter-to-summer length changes resulting from the change in the moisture content of the concrete alone can be obtained from them with considerable accuracy. For example, the range in the observed unit length changes of the short sections, as determined from the expansion and contraction periods shown in the figure, is 0.000205 to 0.000240 for the 60° F. change in slab temperature. Inasmuch as the short sections were structurally intact and were relatively free to move, the length changes are principally those caused by changes in the temperature and moisture content of the concrete. It will be recalled that, earlier in the report, the average value of the thermal coefficient of the concrete was estimated to be approximately 0.0000048 per degree F. From this, the unit length change of the sections for a 60° F. change in temperature can be calculated and applied as a correction to the observed unit length change, yielding the unit length change caused by the change in the moisture content of the concrete.

In this investigation the unit length changes caused by the annual cycle of moisture variations were found to range from 0.000048 to 0.000083, these length changes being opposite in sense and partly compensatory for those caused by the annual cycle of temperature changes. For the Indiana pavement, these values correspond to length changes produced by a 10° to 17° F. change in pavement temperature. The values should be approximately a maximum for the yearly cycle, since, as remarked before, the data were obtained at times when the maximum and minimum amounts of moisture were present in the concrete.

Again referring to data obtained from the 40-foot, plain concrete test section of the Arlington experiment, it was found that seasonal variations in the moisture content of that concrete caused length changes corresponding to a 20° to 40° F. change in slab temperature. Hence, it is evident that the effect of moisture on length changes was less for the reinforced sections in Indiana than for the plain concrete section of the Arlington study. In this comparison, also, it seems quite possible that the reinforcing steel restrained, to some extent, the tendency of the concrete to change in length with moisture change.

As a matter of interest, it is noted that in tests conducted by the Minnesota Department of Highways [4] seasonal moisture variations caused length changes corresponding to slab temperatures that averaged 20° F. Also, it was determined in tests by the Michigan State Highway Department [4] that, for a constant temperature of 72° F., the average unit change in length of plain concrete specimens from an oven-dry to a saturated state was 0.000246, this value being equivalent to a change in temperature of 46° F. It appears that different concretes may vary considerably in this characteristic.

The progressive or permanent changes in the length of several of the longer sections are given in figure 12. These data were obtained at the same temperatures and on the same days as those of the short sections discussed previously. The lengths of the individual bars indicate over-all changes in section length that accompanied a 60° F. winter-to-summer rise in pavement temperature. The solid line drawn through the mid-points of the individual bars defines the progressive changes in the lengths of the sections.

It is apparent from this figure that the lengths of all sections increase progressively with time, and that the magnitude of these progressive increases becomes greater with increase in section length for sections up to approximately 1,000 feet long.

The progressive increases of the long sections are the result not only of the tendency of concrete to grow when exposed to cycles of moisture and temperature change, as was the case of the short, uncracked sections; but also of the tendency of transverse cracks to open (however slightly) with time, and of the influence of subgrade resistance. The fact that the long sections returned so nearly to their original or base lengths during the early cycles of length change indicates that the initial widths of the many transverse cracks that

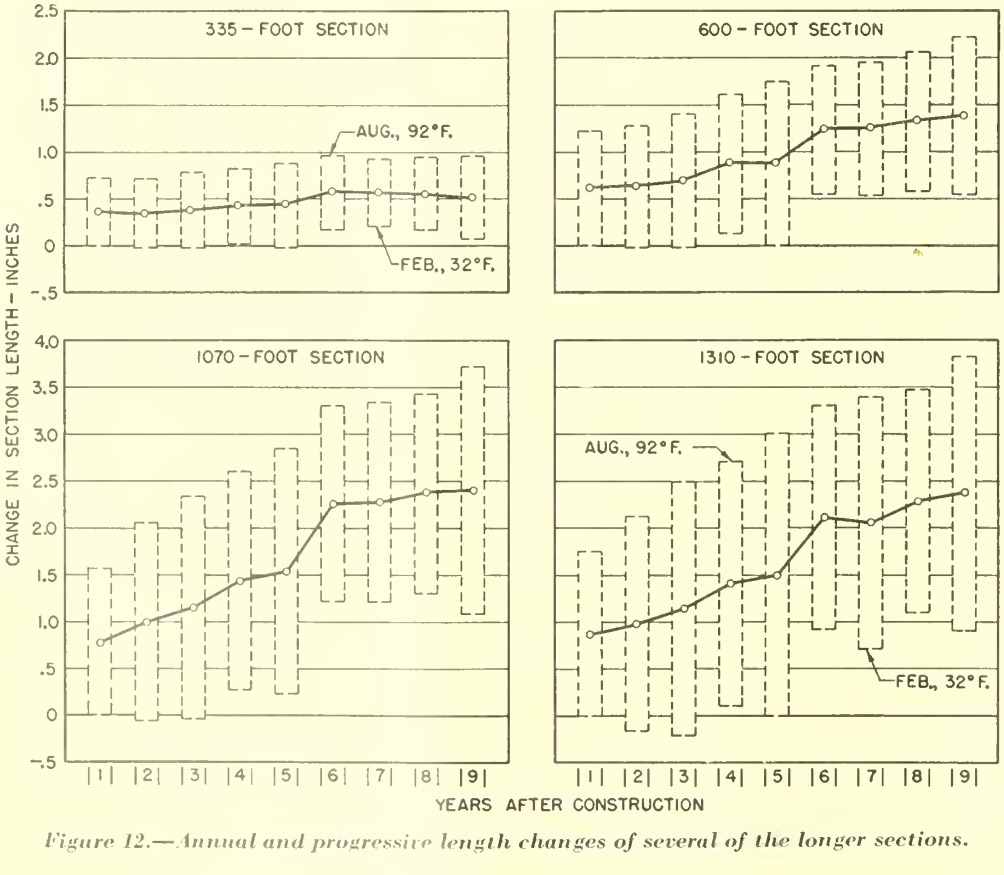


Figure 12.—Annual and progressive length changes of several of the longer sections.

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[3] *The structural design of concrete pavements—part 2,* by L. W. Teller and E. C. Sutherland; PUBLIC ROADS, vol. 16, No. 9, Nov. 1935; and *Application of the results of research to the structural design of concrete pavements,* by E. F. Kelley; PUBLIC ROADS, vol. 20, No. 6, Aug. 1939.

[4] *Investigational Concrete Pavements,* Progress reports of cooperative research projects on joint spacing; Highway Research Board, Research Report No. 3B, 1945.

developed during this period must have been extremely small. Also, when the magnitude of the total increase in section length or growth of these sections is divided by the number of cracks in the section, it is evident that the steel reinforcement has prevented any appreciable opening of the individual cracks.

Part 3.—DEVELOPMENT AND DISTRIBUTION OF CRACKS

Five crack surveys were made over the full length of the experimental pavement during the 10-year period. The first was made shortly after the sections were placed; others were made at the end of the first, third, fifth, and tenth years of service. In addition, during the first 3 years of the life of the pavement, certain representative sections were surveyed at more frequent intervals. In every case the surface of the pavement was subjected to a very careful examination in order that all fractures visible to the naked eye might be detected.

Figure 13, traced from the crack survey sheets, shows the number and position of the cracks that have developed in typical sections during the 10-year period of service. Considerable care was exercised in accurately plotting each crack on the original survey sheets. Because of the fine character of the cracks, it was necessary to outline each crack with keel on the surface of the pavement before plotting on the sheets.

It will be noted from the examples in figure 13 that short sections tend to be comparatively free of fractures. At the end of 10 years, 70 percent of 154 short sections—that is, those whose lengths range from 20 to 120 feet—were still uncracked. As the section lengths increase, however, cracking becomes more prevalent until in the central portion of long sections the crack interval is frequently less than 2 feet.

It may be observed also, from the crack patterns shown by the survey sheets, that: (1) Cracks, although somewhat wavy and irregular, are essentially at right angles to the axis of the pavement; (2) cracks in many instances are not continuous across both lanes, either ending completely or being offset slightly at the center joint; (3) longitudinal cracking has not developed in any part of the pavement; and (4) corner breaks at transverse cracks are very rare.

After 10 years of service the surface condition of the pavement is excellent. With the exception of those in sections reinforced with the 32-pound wire fabric, all fractures have been held closed by the longitudinal steel. The cracks that formed in the sections containing this light fabric were wider initially than those that appeared in the more heavily reinforced sections and, after about 8 years, in several cases the steel crossing them broke, probably from shearing forces, resulting in relatively wide openings and some spalling. In all of the other sections there is no evidence of any form of structural damage to the concrete, with the exception of a very slight raveling of the edges of the cracks, probably due to flexure. It is believed that the fineness of the cracks, especially those in the more heavily reinforced sections, is conducive to distributed interfacial pressure, thus minimizing the possibility of blow-ups and other pressure concentration failures that are sometimes observed at cracks in plain concrete pavement.

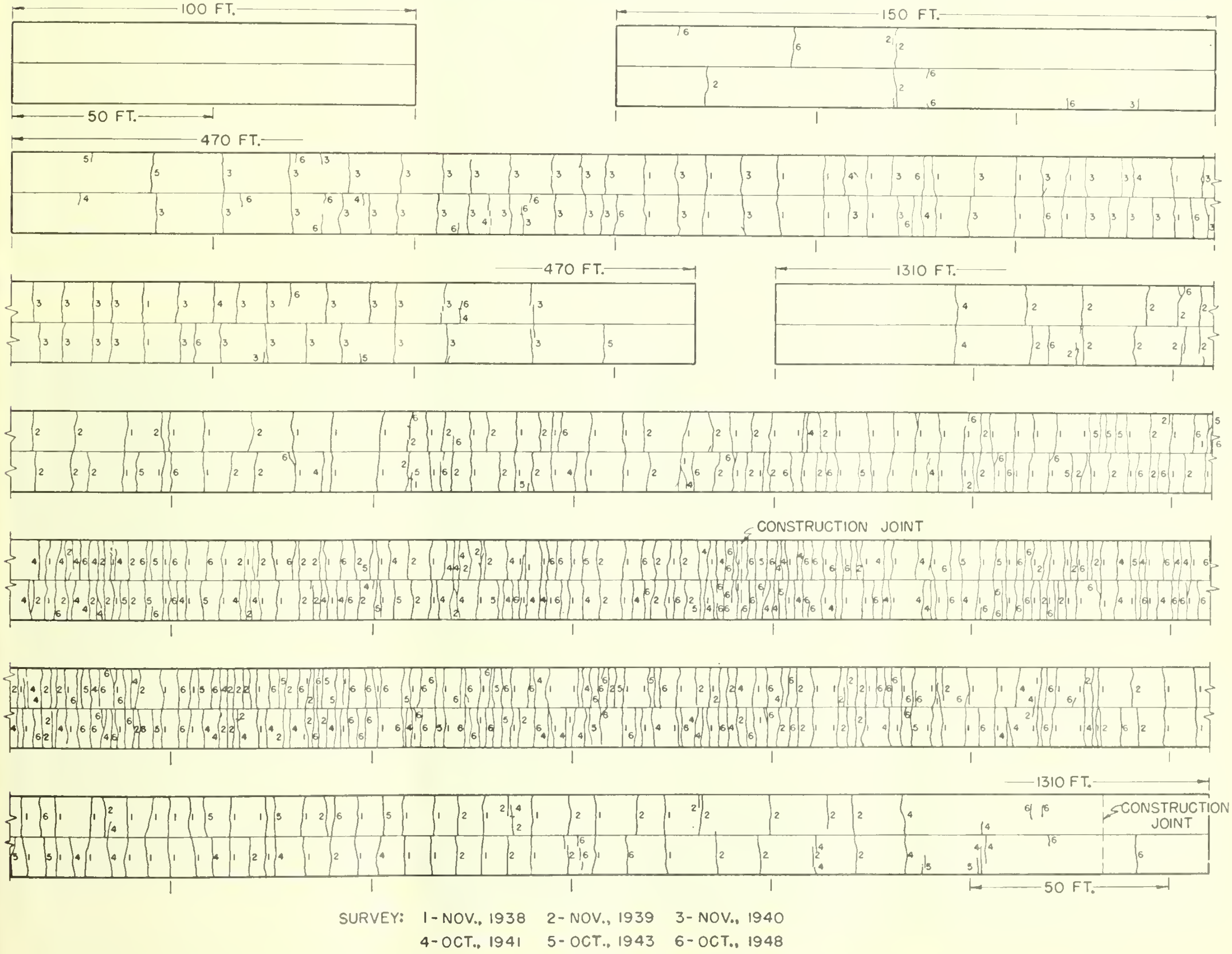

Figure 13.—Typical formation of cracks during the first 10 years of pavement life (sections placed September–October 1938).

Surface Condition at Cracks

Figures 14 and 15 show typical examples of the surface condition of the pavement at several cracks.

Figure 14 pictures the surface condition of the pavement in the vicinity of several of the widest cracks to be found in the central region of the heavily reinforced sections. These cracks are in the right-hand or heavily traveled lane of the pavement and were taken after a 7-year service period. Unfortunately, since the photographs were taken, many of these cracks were inadvertently covered with bituminous material by a maintenance crew. This material did not enter the cracks but did spread over the surface and obscure them.

In figure 15 are shown close-up photographs of the surface condition of the pavement at two cracks; one in the central portion of the longest section of those reinforced with 1-inch diameter rail-steel bars (1.82 percent of longitudinal steel) and the other in a comparable portion of the longest section of those containing ½-inch diameter rail-steel bars (0.45 percent steel). Both photographs were taken after the pavement had been in service for 10 years and show fractures typical of those that appeared in the heavily traveled lane shortly after construction.

The contrast between the surface widths of the two cracks pictured in figure 15 is quite obvious when actually seen in the pavement. When the cracks first formed, those that appeared in the most heavily reinforced sections were almost microscopic, being discernible only by extremely close inspection. However, in the sections with decreasing percentages of reinforcing steel the cracks were, in general, proportionately less frequent and more readily seen; cracks in the lightly reinforced short sections, if present at all, being relatively conspicuous. Over the 10-year period of service the action of traffic and exposure has produced some slight raveling and rounding of the edges of the fractures.

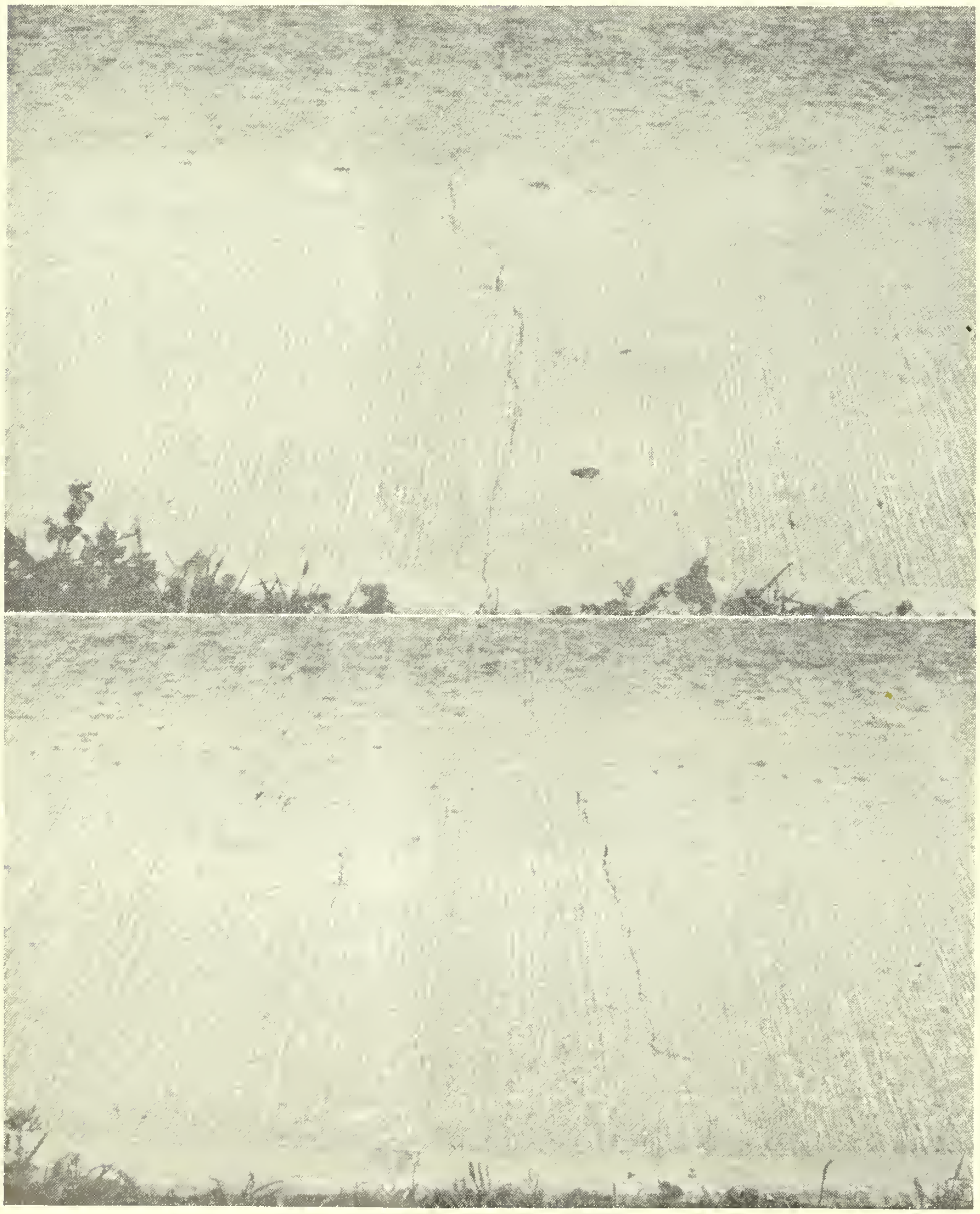
Figure 14.—Surface condition of pavement in vicinity of the widest cracks observed in the central portion of the heavily reinforced sections (heavily traveled lane after 7 years of service).

The preceding discussion related to differences between the surface widths of fractures in sections containing different percentages of longitudinal reinforcement. During the surveys, it was observed further that cracks in the end portion of a given long section generally presented a slightly better surface appearance than those in the central part; and that cracks in the central portion of sections containing a given percentage of steel, but of different lengths, showed some slight evidence of a corresponding difference in surface widths, those in the central part of the longest section of each group apparently being wider than those in the central part of the shortest section. It seems reasonable that this should be so.

Quantitative Measurements

At the end of 10 years, quantitative measurements of the surface widths of cracks, which included raveling and rounding of their edges, were made in the following manner: Starting at the edge of the pavement the width of a segment of crack about 3 feet long was carefully examined and a width measurement made at a point judged to be average. A similar measurement was made on each of two additional 3-foot segments of the same crack, thus covering one lane width. The average of the three measurements was considered to be the average surface width of the crack for the particular lane. All measurements were estimated to the nearest 0.01 inch. It is realized that this procedure does not establish an exact value for the surface width of an individual crack, but it is believed that the averages of a number of such measured values have significance in relative comparisons.

Measured values of the surface widths of cracks obtained in the manner described are given in table 4. The values shown, for each percentage of steel, are of fractures that developed at an early age in the central area of the longest section reinforced with either rail-steel or billet-steel bars. Hence, the computed maximum steel stress was either 45,000 or 55,000 pounds per square inch. An average width value represents the combined average of 15 or 20 cracks, measured in sections containing both rail-steel and billet-steel bars. All data were obtained in the fall of the year when the mean pavement temperature was 58°–60° F.

Table 4.—*The surface width of cracks (central portion of longest section for each percentage of steel)*

Percentage of longitudinal steel in section [1]	Surface width of cracks in lane carrying—			
	Heavy traffic		Light traffic	
	Average	Range	Average	Range
Percent	*Inches*	*Inches*	*Inches*	*Inches*
1.82	0.053	0.02–.11	0.020	0.01–.03
1.02	.078	.03–.15	.032	.02–.05
.45	.104	.07–.18	.038	.02–.06
.26	.117	.09–.15	.038	.02–.07

[1] Calculated maximum stress in steel is either 45,000 or 55,000 pounds per square inch.

The comparisons available in the table show that the surface widths of the cracks tend to increase with a decrease in the amount of longitudinal reinforcement. For example, the average measured width of the cracks in the heavily traveled lane of the selected sections reinforced with 0.45 percent steel is approximately twice the average width of those in sections containing 1.82 percent of steel. The influence of traffic on the surface width of the fractures is also evident, but this effect will be discussed later in the report.

As previously mentioned, the values of table 4 are for sections containing rail- and billet-steel bars. Measurements of the surface widths of cracks also included fractures in the longest section of each group reinforced with the 91- and the 149-pound welded-wire fabric. The data from these measurements are concordant with those from the sections reinforced with rail- and billet-steel bars. The average surface width of the cracks in the section containing the 91-pound fabric was found to be appreciably greater than that of the section reinforced with the 149-pound fabric.

Referring to the range in the average surface width of individual cracks (table 4), it is apparent that the maximum is, in some cases in the heavily traveled lane, slightly more than one-eighth inch. Also, the width of a crack at isolated points along its length was often observed to be considerably greater than its average width, because of localized raveling. The maximum values at such points were 0.3 and 0.7 inch, respectively, for sections reinforced with 1.82 and 0.45 percent of steel. It should be kept in mind, however, that the depth of raveling along the lengths of all cracks was estimated to be never more than one-eighth inch and may be considered superficial.

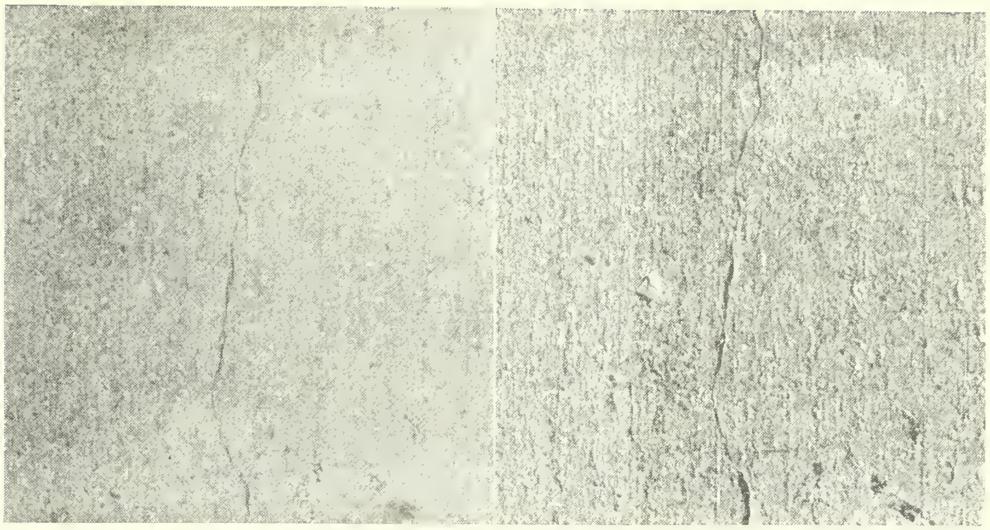

Figure 15.—Surface condition of cracks typical of those that developed at an early age in the central portion of the longest section reinforced with: (left) *1-inch diameter bars (1.82 percent steel); and* (right) *½-inch diameter bars (0.45 percent steel). These were in the heavily traveled lane after 10 years of service.*

A limited amount of supplementary data on the surface widths of fractures, other than those given in table 4, were obtained by measurements in the end and central areas of the 1,310-foot section, in order to establish a comparison of crack widths in those regions. It was found that the surface widths of cracks in the central portion of the 1,310-foot section averaged about twice the width of those near the ends. Measurements were also made of the surface widths of cracks that had developed in the central part of the 600-foot section reinforced with 1.82 percent of steel and such widths averaged about one-half the width of those that formed in the central part of the 1,310-foot section containing the same percentage of reinforcement.

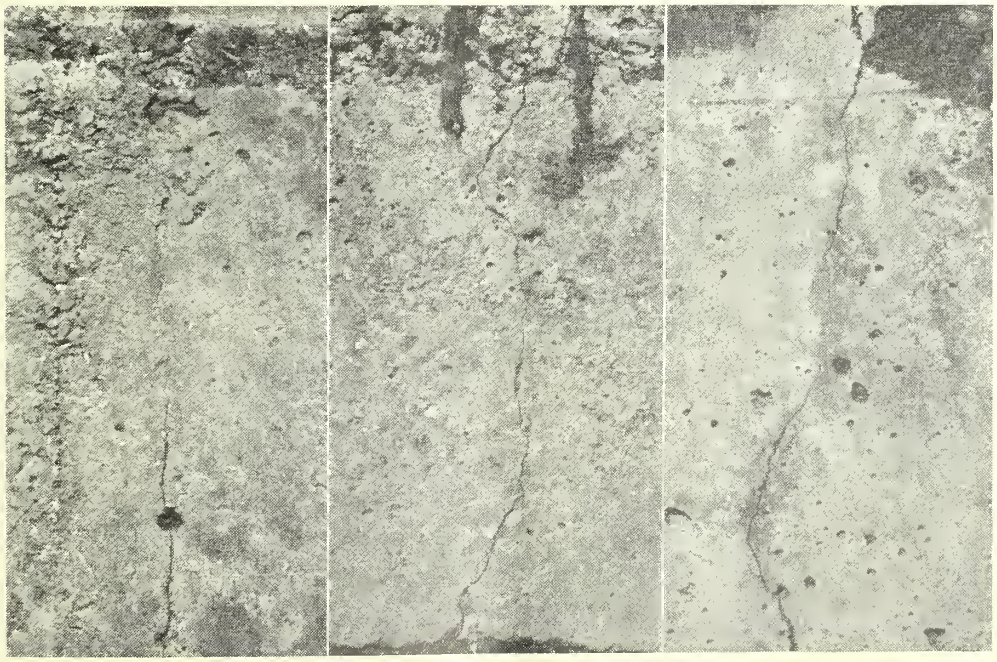

Figure 16.—Edge (vertical face) condition of cracks typical of those that developed at an early age in the central portion of the longest section reinforced with: (left) *1-inch diameter bars (1.82 percent steel);* (center) *½-inch diameter bars (0.45 percent steel);* and (right) *91-pound wire fabric (0.24 percent steel). These were in the heavily traveled lane after 10 years of service.*

Real Widths of Cracks

Figure 16 shows close-up photographs of several cracks as observed at the vertical face at the edge of the pavement. These photographs, taken in 1948, show fractures that occurred at an early age in the central area of the longest section of each group reinforced with 1-inch diameter rail-steel bars, ½-inch diameter rail-steel bars, and a 91-pound wire fabric, the percentage of reinforcement values being 1.82, 0.45, and 0.24, respectively.

Cracks, such as those pictured in figure 16, were almost imperceptible when they first appeared, being visible throughout the depth of the slab only after a drying period following a wetting of the concrete. With time, however, they have opened progressively a very small amount and their edges have raveled slightly.

At the end of 10 years of service, measurements were made of the edge-face widths of a number of cracks (located in the central portions of the sections mentioned above) in order to obtain values of the real widths of the cracks themselves; width values which, unlike those taken on the surface of the pavement, did not include raveling and rounding of the crack edges. A 40-power shop microscope with a 0.001-inch graduated scale was used to make the measurements, the instrument being focused into the opening of the fracture to eliminate errors caused by surface conditions at the crack edges.

The data obtained from this study of crack widths in the slab edges are given in table 5. Each average value of the table is the average for five cracks that developed early in the life of the pavement. The computed maximum steel stress at the site of these cracks is 55,000 pounds per square inch. All measurements were made at the mid-depth of the slab and in the fall of the year when the mean pavement temperature was 73°–74° F.

The values shown indicate that the real widths of the cracks, like their surface widths, increase with a decrease in the percentage of longitudinal reinforcement. For example, the

Table 5.—*The real width of cracks (central portion of longest section for each percentage of steel)*

Percentage of longitudinal steel in section [1]	Width of cracks in lane carrying—			
	Heavy traffic		Light traffic	
	Average	Range	Average	Range
Percent	*Inches*	*Inches*	*Inches*	*Inches*
1.82	0.004	0.002-.007	0.002	0.001-.003
.45	.011	.007-.018	.009	.007-.010
.24	.013	.005-.018	.010	.006-.013

[1] Calculated maximum stress in steel is 55,000 pounds per square inch.

average width of the fractures in the heavily traveled lane of the selected section reinforced with 0.45 percent of steel is nearly three times the average width of those in the same lane of the section containing 1.82 percent of steel. A comparison of the data in tables 4 and 5 shows that the surface width of cracks increases under the same conditions that cause an increase in real width. It is apparent, also, that the surface width of a given crack is many times greater than its real width.

Longitudinal reinforcement is in continuous bond with the concrete until the first transverse crack develops. When this happens the amount of opening of the crack will depend upon the total elongation of the steel which crosses it. This elongation is, in turn, dependent upon the length that is free to elongate as affected by the bond between the steel and the concrete; and upon the magnitude of the direct tensile stress in the steel, also dependent upon bond conditions.

In this investigation neither the length over which the steel was not in bond nor the magnitude of the tensile stress in the steel could be determined.

However, it is of interest to examine the crack-width data on the basis of the amount of longitudinal steel present, as shown in table 5. Presumably, at the time of the crack-width measurements the same steel stress was active in the central region of all sections listed in the table. When compared in this way it will be found that for both the heavily traveled and the passing lanes, the average crack width increases directly with a decrease in the percentage of longitudinal steel.

Also of interest is the fact that in the longer sections the surface widths of cracks, and presumably their real widths also, were less in the end than in the central areas of the sections. This is as would be anticipated, since the tensile stress in the longitudinal bars would be expected to decrease as the end of a section is approached.

Effect of Traffic

In connection with the study of cracking, an opportunity has been afforded to observe the effect of traffic on the development and condition of the cracks. It will be recalled that the experimental two-lane pavement is one-half of a divided highway; consequently, the right-hand lane carries the greater number of vehicles and practically all of the heavy trucks, the left-hand lane being used largely for passing. Also, it is mentioned again that the experimental sections, part of U S 40, are subjected to a relatively high frequency of heavy traffic loads.

Although a survey made soon after completion of the pavement showed equal cracking in both lanes, at the end of the first year 51.2 percent of the total number of cracks were found to be in the right-hand lane of the pavement. This percentage value had increased to 52.7 and 53.0 percent at the end of the fifth and tenth years, respectively. Thus, it appears that repetition of traffic loads has exerted a slight but only a slight influence on the development of transverse cracks. Since approximately two-thirds of the total or present number of cracks formed during the first year, when only 51.2 percent formed in the right-hand lane, the effect of traffic repetition on subsequent cracking is somewhat more pronounced than is indicated by the preceding percentage values.

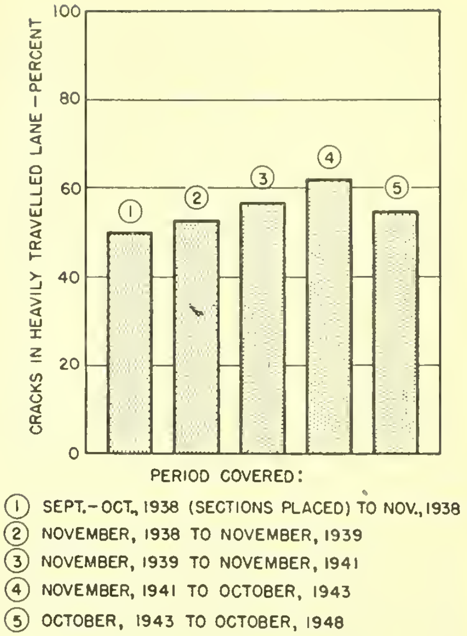

Figure 17.—Effect of traffic on the amount of cracking.

Figure 17 was prepared to show the influence of traffic on cracking during specific periods of the life of the pavement. For the periods indicated by the circled numbers, each individual bar represents the number of cracks that formed in the heavily traveled right-hand lane expressed as a percentage of those that formed in both lanes of the pavement. It will be noted that an equal number of cracks appeared in both lanes of the pavement within the first month or two after construction. During each subsequent period a progressively greater number of cracks formed in the heavily traveled lane than in the passing lane until a maximum value of 62 percent was reached for the period covering the third to fifth years of pavement life. During the last 5 years about 55 percent of all new cracks developed in the right-hand lane of the pavement.

Also, as will be seen by an examination of the data in tables 4 and 5, traffic has had an

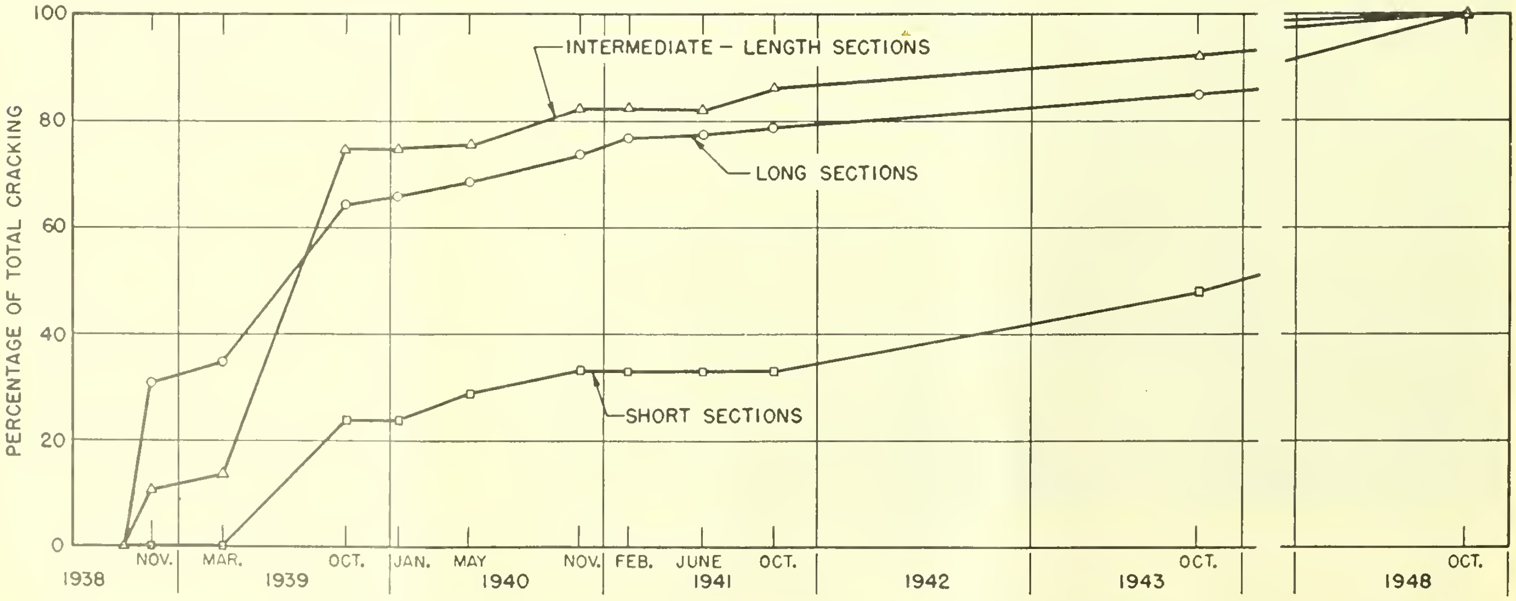

Figure 18.—Rate of crack development during the first 10 years of pavement life.

appreciable effect on both the surface and the real widths of the transverse cracks. Comparisons of the average widths of cracks in the right-hand lane with those of companion cracks in the passing lane are given in table 6.

It is apparent that the heavier traffic using the right-hand lane has produced more extensive raveling and other superficial damage at the crack edges and a wider separation of the fractured faces than the lighter traffic on the left-hand lane. This effect of traffic is naturally more pronounced in the case of the surface widths of the cracks.

Rate and Distribution of Cracking

Figure 18 shows the manner in which cracking has developed with respect to periods of time. In this figure the sections were grouped according to length, as short, 20–120 feet; intermediate-length, 120–470 feet; and long, 470–1,310 feet.

Thirty-one percent of the total or present number of cracks in the long sections and 11 percent of those in the intermediate-length sections appeared within approximately 1 month after construction. Few cracks occurred during the first winter, none in the short sections. However, the rate of cracking was quite high for all groups during the period that followed, a period that included the interval between late March and late October of the first year.

The survey made at the end of this first year of service showed 65 percent of the present cracking had developed in the long sections, 75 percent in the intermediate-length sections, and 25 percent in the short sections.

On the basis of all transverse cracks that have developed during the 10-year period of service, it is of interest that 67 percent had appeared by the end of the first year. After the first year the rate has been quite low and, in general, rather uniform. Between length groups the highest rate has been in the short sections.

In figure 19 is shown the distribution of cracking for representative sections, expressed as the number of cracks per 50 feet of section. The data indicate that the number of cracks per 50-foot increment increases from a minimum value at the end of a section to a maximum value in the central area, in a generally normal frequency distribution pattern; and that the maximum values, as found in the central area of the sections, increase progressively with increase in section length.

It will be noted that the symmetry of cracking in the experimental sections is not only indicative of structural uniformity, but also implies that the nonuniformity of the elevation changes that developed in the pavement, as mentioned earlier in the report, apparently had little effect on the formation of cracks.

The manner in which cracking developed in the sections is, in some respects, shown to better advantage in figure 19 than in figure 18, especially since the distribution of cracking for the various time periods is given in the latter figure.

The magnitude and distribution of the cracking that appeared within approximately 1 month after construction of the sections are shown as the first time period. During this period no cracks were found in sections having lengths of 210 feet or less, and only a limited number in the central portion of sections with lengths between 270 and 360 feet; but a considerable number were found in the 600- and 1,070-foot sections at some distance from the ends. Since the cracking during this period appeared only in the central areas of the longer sections, it is believed that it had its origin primarily in the tensile stresses induced by subgrade resistance during shrinkage of the sections either from loss of moisture, decrease in pavement temperature, or both.

Table 6.—*Comparison of average widths of cracks in right-hand lane with widths of companion cracks in left lane*

Percentage of longitudinal steel in section	Ratio of crack width in right-hand lane to that in left lane.
SURFACE CRACK WIDTH	
0.26 .45 1.02 1.82	3.1:1 2.7:1 2.4:1 2.7:1
REAL CRACK WIDTH	
0.24 .45 1.82	1.3:1 1.2:1 2.0:1

The second time period covers the first winter after construction. The survey at the end of the winter indicated that sections having lengths of 210 feet or less were still uncracked and that only a small amount of cracking, spottily distributed, had developed

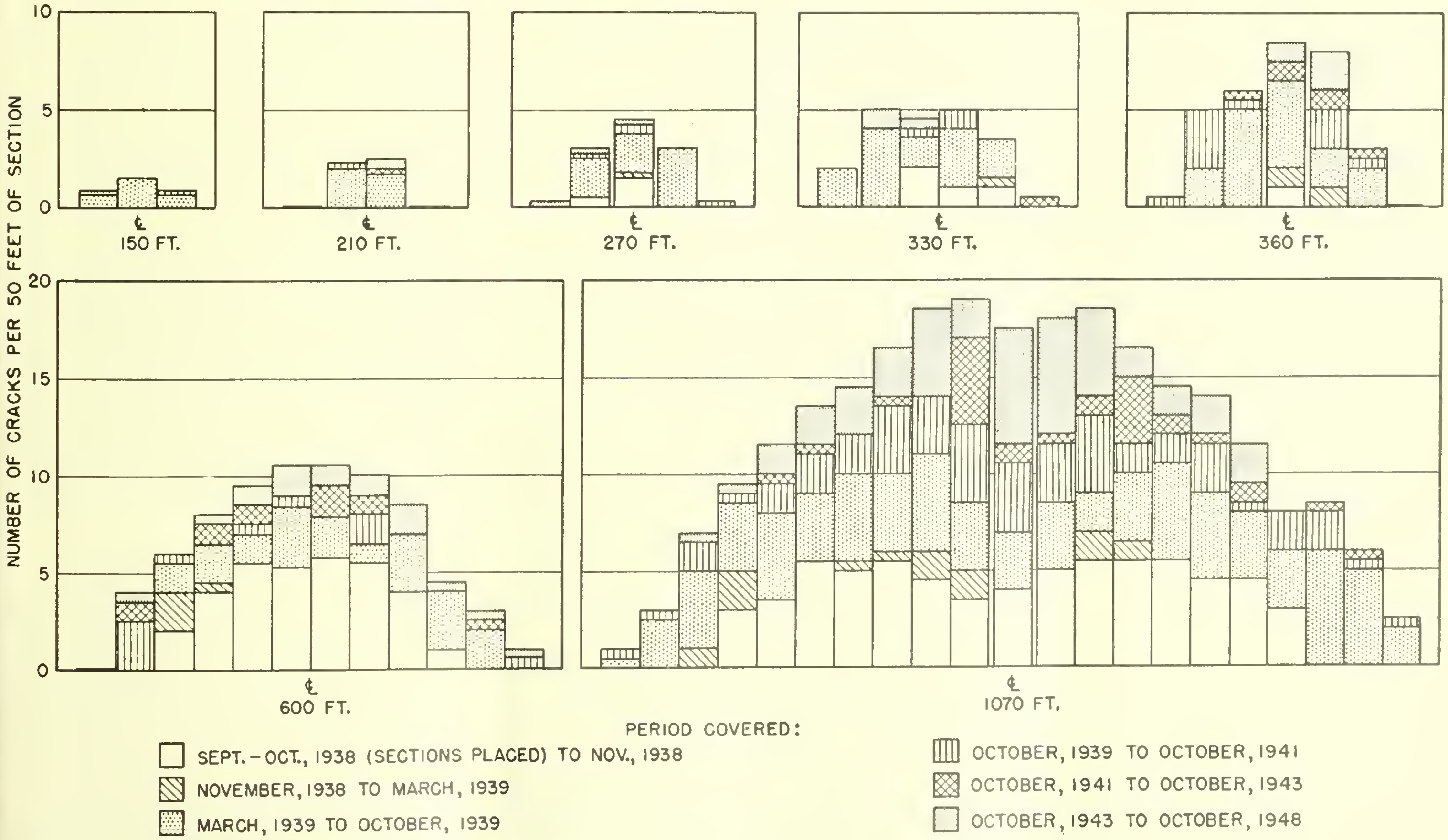

Figure 19.—Distribution of cumulative cracking per 50 feet of section length in the first 10 years of pavement life.

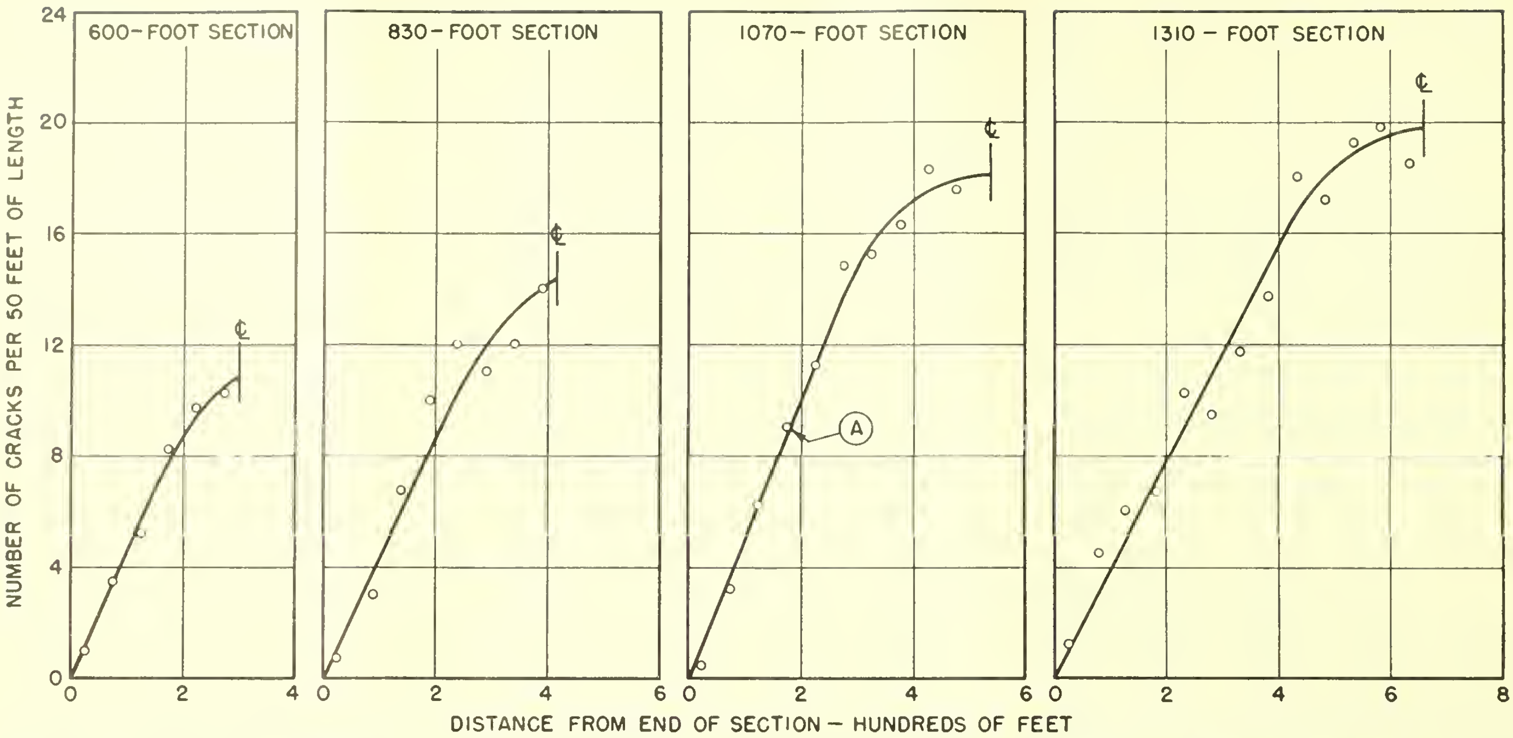

Figure 20.—Frequency distribution of cracking at the end of 10 years for sections reinforced with 1-inch diameter rail-steel bars (1.82 percent steel); average of both lanes.

in sections having lengths equal to or greater than 270 feet. The relative absence of crack development during this period indicates that the nonuniform changes in pavement elevation caused by frost penetration of the subgrade had little influence upon cracking; and that tensile stresses from subgrade resistance were no greater during the winter period than during the preceding fall. This supports the conclusion drawn from the data of figure 9.

In the third time period, between late March and late October of the first year, a noticeable change occurred in the crack development in all of the sections. In fact, a large percentage of the cracks now present in sections having lengths of 360 feet or less, and in the end areas of the longer sections, formed sometime during this period. Such cracking is believed to have been caused primarily by stresses induced by restrained warping. Unfortunately, the pavement was not surveyed in midsummer so it is not possible to determine more closely the part of this time period during which cracks formed. However, it is suspected that the cracks developed in large part during late spring and early summer when warping stresses are generally highest for the year. The fractures that formed at some distance from the ends of the longer sections may have resulted, also, from stress combinations existent in early fall when the sections were contracting after having attained their maximum annual unrestrained lengths.

During the fourth time period, which covers the second and third years after construction, the development of new cracks was confined primarily to the central areas of the long sections. The relatively small number of fractures that appeared during this period and during the succeeding fifth time period (fourth and fifth years) greatly reduced the rate of crack development. Within the sixth and last time period of this study, from the fifth through the tenth years of pavement service, the greatest number of cracks again have formed in the central areas of the long sections, suggesting a continued high stress condition in those regions.

Crack Frequency Patterns

Frequency distribution curves for the cracking that existed at the end of 10 years in the four sections comprising the group reinforced with 1.82 percent of longitudinal steel (1-inch rail-steel bars) are shown in figure 20.

The ordinate values represent the number of cracks per 50 feet of section and the corresponding abscissas are distances from the end of the section to the centers of the 50-foot lengths to which the ordinate values apply. For example, at point *A* in the figure there are nine cracks in the 50-foot length which lies between 150 and 200 feet from the end of the 1,070-foot section.

It is apparent that: (1) For some distance, beginning at the end of a section, the crack frequency for successive 50-foot increments increases directly with increase in distance; (2) the length over which the linear relation holds increases progressively with increase in section length; and (3) the slopes of the linear portions of the curves appear to be nearly the same for the different section lengths. It is believed that the frequency of cracking in the sections reflects, to a considerable extent, the stress distribution in the longitudinal steel as induced by subgrade resistance.

Frequency distribution curves were constructed for all sections having lengths equal to or greater than 150 feet. From these curves the maximum cracking frequency value (number of cracks per 50 feet of section in the central area) was determined for each section. Figure 21 shows, for each of the three types of reinforcement, the relation established by plotting such frequency values against the corresponding section lengths. In order to show possible effects of the stresses in the steel, the maximum computed steel stresses are indicated by symbol.

It is apparent that the maximum cracking frequency increases with an increase in section length, the relation being nearly linear for section lengths between 400 and 1,000 feet. For section lengths greater than about 1,000 feet the curves depart from linearity, indicating that a condition of complete restraint is being approached. This suggests that sections having lengths somewhat greater than 1,000 feet, possibly the 1,700–1,800-foot length mentioned in the discussion of figure 9, would develop complete restraint in the central region and that sections of this length or greater would have equal maximum cracking frequencies irrespective of their over-all lengths.

The 10-year data indicate that an interval between cracks in this region of complete restraint might be expected to be approximately 2.0 to 2.5 feet.

Effect of Reinforcement Type

The data shown in figure 21 indicate that the type of reinforcement has only a slight effect on maximum cracking frequency. For example, within the length range of sections containing welded-wire fabric, the maximum cracking frequency values are only slightly less than those for sections of comparable length reinforced with billet- or rail-steel bars. Comparing the bar-reinforced sections, it appears that the maximum cracking frequency values are slightly greater for billet- than for rail-steel bars (at 1,000 feet these values are 18.8 and 17.0, respectively).

On the other hand, it will be noted that all symbols denoting the various magnitudes of computed steel stresses fall very close to the mean curves, indicating that, within the range of steel percentages in sections of common length, a variation in the amount of steel is not accompanied by a corresponding variation in maximum cracking frequency.

Because of conservative design assumptions, the relation between amounts of reinforcing steel and the section lengths are such that the steel has, in all probability, never been stressed beyond its elastic limit. Therefore, if reinforcement which is adequate for a given section length, say 400 feet, had been used for the entire range of section lengths, the relations shown in figure 21 would not obtain and differences due to variations in the maximum steel stresses might have appeared. The longer sections would probably either subdivide due to breakage of the steel or contain fewer but wider cracks as a result of inelastic deformation of the steel.

Another analysis of the data that is of interest is shown in figure 22 in which the average slab length, after 10 years of service, is plotted against section length as constructed. In this figure separate curves are given for each of the three types of reinforcement, and the maximum steel stresses as computed during the designing of the sections are indicated by symbol. Slab length is defined as the distance between transverse cracks or joints, all joints being considered as cracks. Each point defining the curves is an average value of either two, four, or six sections.

Although the points defining the curves appear to be somewhat erratic for sections up to approximately 200 feet in length, it is believed that this is a statistical effect caused by the relatively small number of cracks in sections of these lesser lengths.

It is apparent that the three curves of figure 22 follow the same general pattern; that is, the average slab length increases with an increase in section length until a peak value is reached, beyond which there is a rapid decrease in average slab length that becomes more gradual and finally approaches a constant value for the longer sections. In the case of the 1,310-foot section the present value of the average slab length is 4.2 feet. At the end of the first and fifth years this value was 7.0 and 5.1 feet, respectively.

The type of reinforcement has quite an obvious effect on the relation between section length and average slab length, especially in the case of the shorter sections. The greatest average slab length for the sections containing welded fabric is 109 feet, which was reached at an optimum section length of 135 feet. In the case of the billet-steel bars the value is 97 feet, attained at an optimum section length of 115 feet; but for the rail-steel bars this value is only 48 feet, the corresponding section length being 90 feet.

The differences in the peak slab-length values and corresponding optimum section lengths for the three types of reinforcement cannot be fully explained. There are, however, two conditions that may have had some influence on the data shown. First, in that part of the curves which pertains to the shorter sections, there are fewer points defining the curve in the case of the rail-steel bars than in the case of the other types of reinforcement. Second, in all sections reinforced with welded fabric and in all sections, except one, with lengths of 270 feet or less reinforced with billet-steel bars, the coarse aggregate used in the concrete consisted of a mixture of small-size gravel with large-size crushed limestone; whereas in sections reinforced with rail-steel bars the coarse aggregate used in the concrete was entirely the crushed limestone. There is no other evidence, however, that the difference in coarse aggregate mentioned affected in any way the behavior or present condition of the sections.

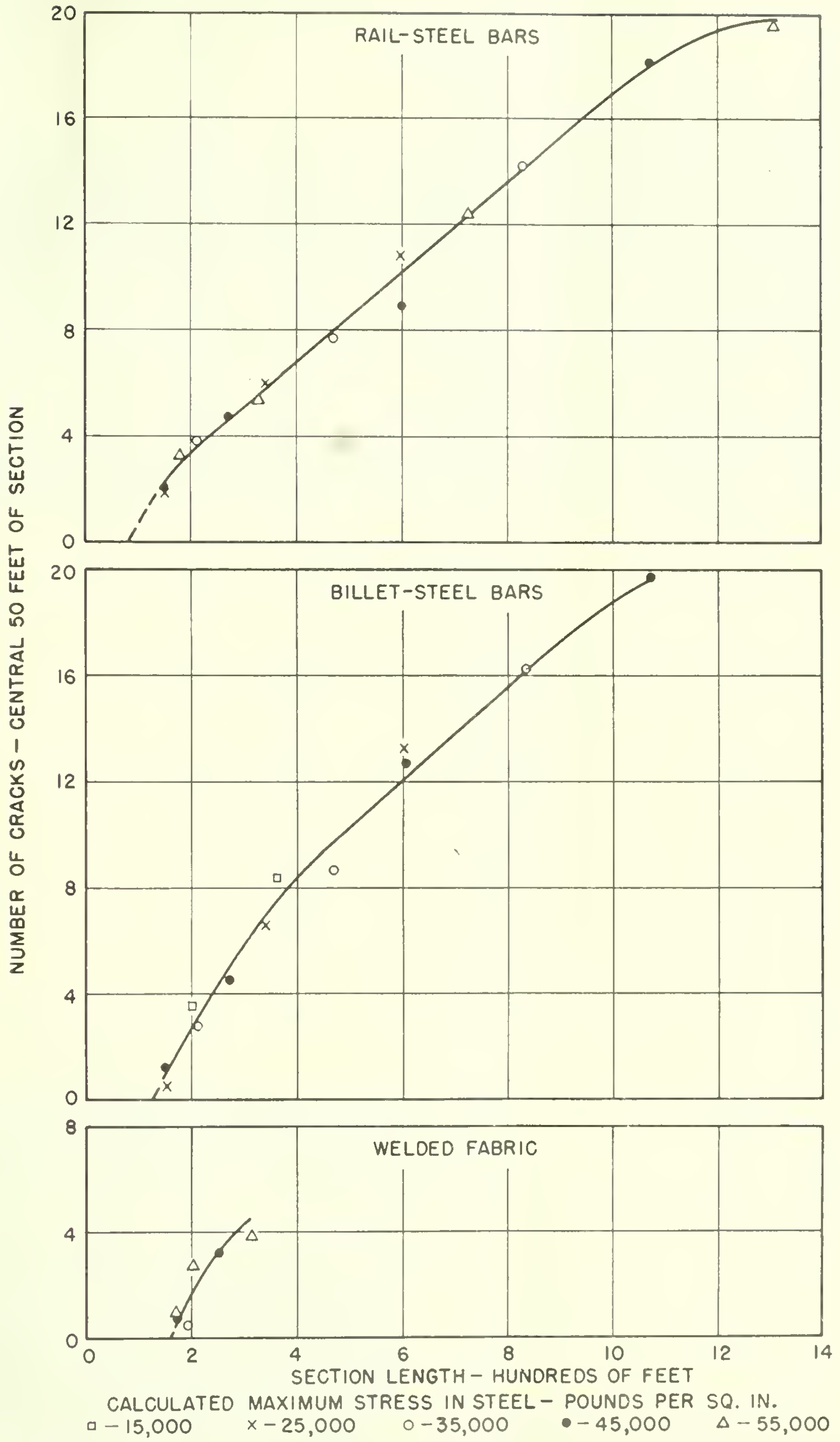

Figure 21.—Effect of type of reinforcement and of calculated maximum steel stress on the relation between section length and maximum cracking frequency at age of 10 years.

The data of figure 22 mean that, under the conditions obtaining in this experimental pavement, the longest average slab lengths are found at the so-called "optimum" section lengths. Hence, if one were interested only in a minimum number of transverse cracks and joints, these data suggest that in reinforced concrete pavements the transverse joints be spaced approximately 100 feet apart. However, as this investigation strikingly shows, a longer section with many transverse cracks can continue to be a strong, durable structural unit after many years of heavy traffic service if it contains an adequate amount of longitudinal reinforcement.

In the relations shown in figure 22, the maximum computed steel stress values apparently have no influence on the amount of cracking in a given section length. This is concordant with the relations shown in figure 21.

Part 4.—BEHAVIOR OF THE SPECIAL SECTIONS

The four special 500-foot sections containing weakened-plane warping joints at 10-foot intervals have been subjected to the same close study as have the regular sections.

It will be recalled that in each of the four special sections, relatively light welded-fabric reinforcement was placed continuously through all of the weakened-plane warping joints over the 500-foot section length. The bond between the steel and the concrete was destroyed purposely for a distance of 18 inches on each side of each joint by omitting two transverse wires, one on either side of the joint, and by greasing the longitudinal wires over the 36-inch length. In addition to the continuous reinforcement, shear bars consisting of ¾-inch diameter dowels 18 inches long, spaced 12 inches center to center, were placed across the warping joints in one-half of each of the four sections.

The distinguishing features of the four 500-foot special sections are as follows:

Section 1.—Weakened-plane joints are of the submerged type and the reinforcement weighs 91 pounds per 100 square feet.

Section 2.—Same as section 1, except that the reinforcement weighs 45 pounds per 100 square feet.

Section 3.—Weakened-plane joints are of the surface-groove type and the reinforcement weighs 91 pounds per 100 square feet.

Section 4.—Same as section 3, except that the reinforcement weighs 45 pounds per 100 square feet.

Through the design features of these special sections it was proposed to develop information on the practicability of a pavement design in which transverse crack control was obtained by means of relatively short slab units (10 feet) with pavement continuity obtained by the use of continuous longitudinal reinforcement. Other information sought pertained to the amount of longitudinal steel necessary to resist the tensile forces created by subgrade resistance in a section of this length; the value of the design feature in which bond was deliberately destroyed for 18 inches on either side of the joint; and the necessity for protection of the longitudinal reinforcement against shear in the transverse joints by means of dowels used to develop shear resistance. It was thought that such protection probably would be necessary because of the relatively large joint opening expected from elastic elongation of the longitudinal steel over the 36 inches of unbonded length at the transverse joints.

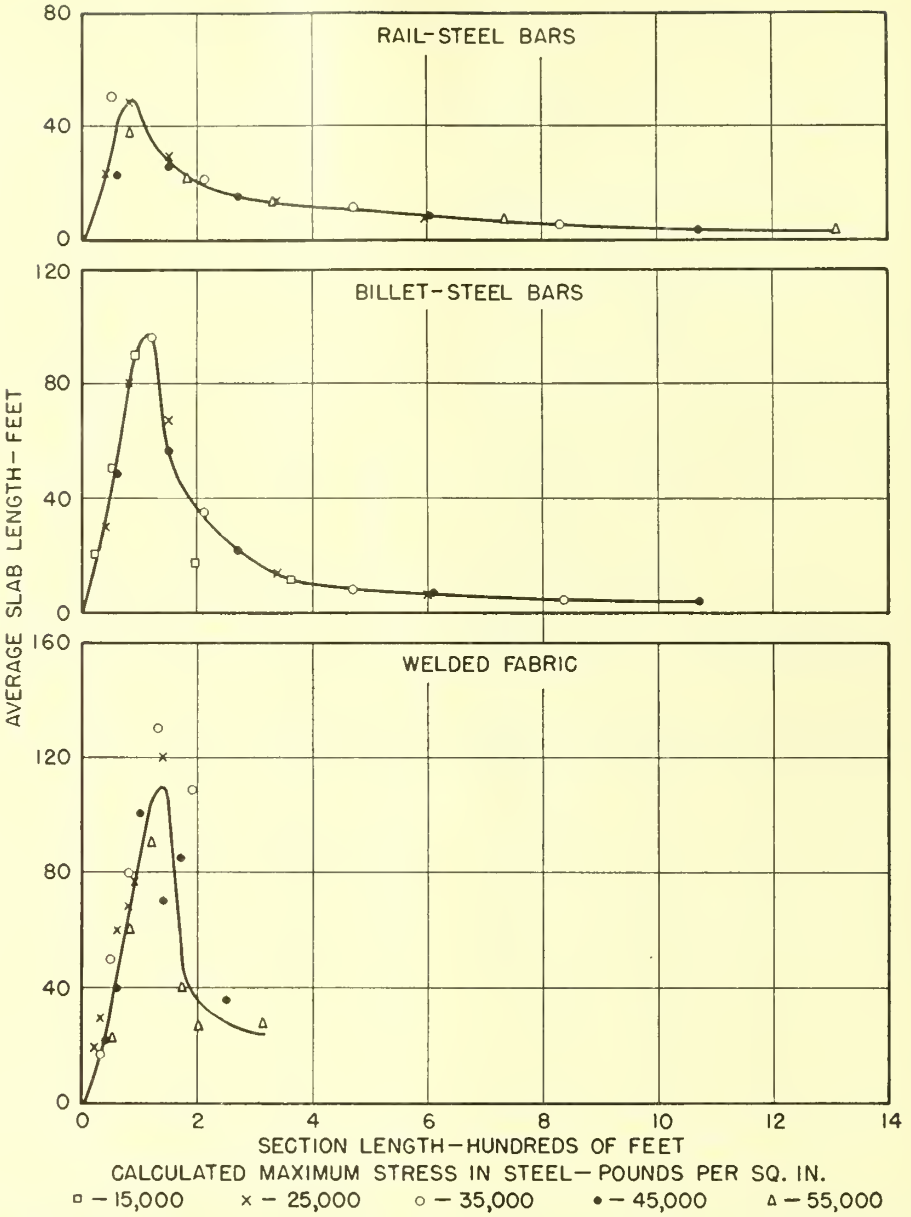

Figure 22.—Effect of type of reinforcement and of calculated maximum steel stress on the relation between section length and average slab length at age of 10 years.

During a drop in pavement temperature, a continuously reinforced section naturally attempts to contract about the center of the section length. At the same time, the individual segments or slab units of the section are attempting to contract about their individual centers. The amount that these individual segments contract should equal the elongation of the steel crossing the fractures that define their lengths. This elongation of the steel is dependent upon the magnitude of the stress induced in it by resistance as the segments tend to move over the subgrade; and upon the length over which the bond between the steel and concrete is destroyed—that is, the length over which this stress is effective.

Thus, by subdividing the 500-foot special sections into 10-foot slab lengths so that during a large temperature drop the contractive length change of an individual slab unit would be relatively small, and by breaking the bond for 36 inches at each separation between slabs so that the elongation of the steel could be relatively great without exceeding the elastic limit, it seemed that a certain degree of control over the movement of a section should be gained without rupturing the reinforcing steel. For example, during a sudden drop in pavement temperature when subgrade resistance is relatively great, the continuous reinforcement would simulate a steel spring at each transverse joint—elongating and permitting the slab units to contract about their individual centers, and subsequently contracting and drawing the units together as the subgrade resistance decreased.

Failures at Joints

From the standpoint of design, all of the special sections behaved satisfactorily during

Table 7.—*Steel failures at joints in the special sections at the end of 10 years*

Joint type	45-pound fabric	91-pound fabric
Surface type:		
With dowels	0	0
Without dowels	9	2
Submerged type:		
With dowels	2	0
Without dowels	4	1

the first 3 years of pavement service. Then the reinforcing steel began to fail at the joints. During the condition survey at the end of 3 years two breaks in the reinforcing steel were discovered, both at joints in the sections containing the 45-pound wire fabric and both at joints without shear bars, one of the breaks being only 60 feet from the end of a section. After 5½ years the reinforcement was either found to be broken or suspected of being broken at seven of the joints. All of these failures developed at joints without shear bars and in the sections reinforced with the 45-pound wire fabric. At the end of 10 years the reinforcing steel was either broken or elongated beyond its elastic limit at 18 of the joints. The distribution of these steel failures is given in table 7.

From table 7 it will be noted that 16 failures were at joints without shear bars; 15 failures developed in sections reinforced with the 45-pound wire fabric; 7 and 11, respectively, were found in sections constructed with the submerged and the surface-groove type of joints; and none occurred in the halves of the sections provided with shear bars at the joints and containing the 91-pound wire fabric. The effect of such failures will be discussed in parts 5 and 6 of this report.

The fact that all of the earlier failures of the reinforcing steel and approximately 90 percent of those now present occurred at joints having no shear bars indicates that shearing forces caused by loads passing over the joints were primarily responsible for the steel failure. However, it is possible for a progressive separation to develop at a joint and eventually overstress the reinforcing steel. Infiltration of solid material would, in time, cause a permanent opening of the joint and dissipate all or part of the elastic elongation of the steel in the 36 inches of unbonded length across the joint. Subsequently, during contraction periods, the steel, if small in amount as in the 500-foot special sections, might be subjected to direct tensile stresses sufficiently large to cause failures. Such action might account for the two cases of steel failure observed at the joints provided with shear bars. These developed after the pavement had been in service for 8 years. Both occurred at some distance from the ends of sections containing the 45-pound wire fabric and both were at joints of the surface-groove type.

Surface Condition of Weakened-Plane Joints

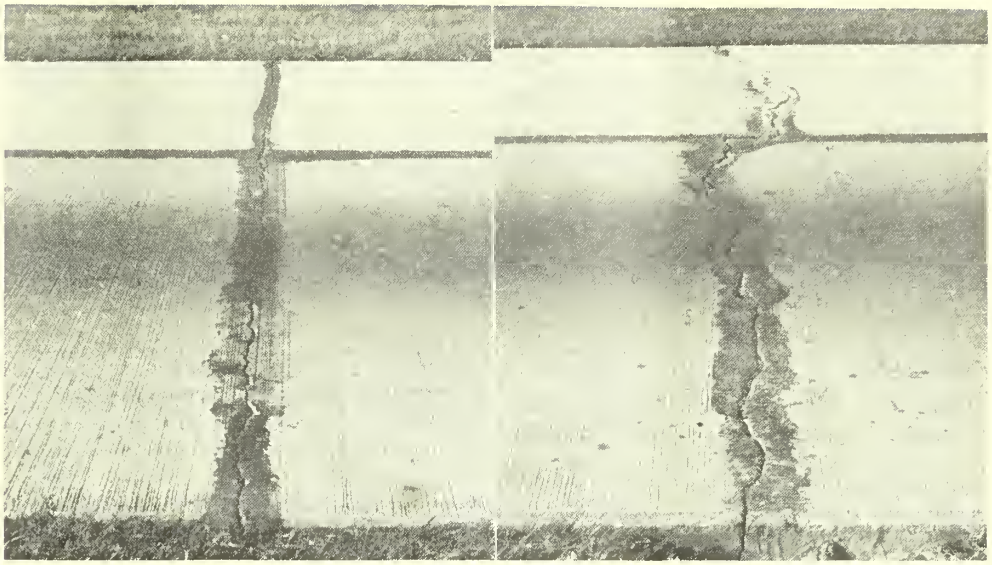

Figure 23.—Two of the submerged-type, weakened-plane joints after 10 years of service: Extreme cases of straight and irregular cracking over the bottom parting strip.

Figure 23 shows photographs, taken after 10 years, of two of the submerged type, weakened-plane joints at which the reinforcing steel was unduly inelastically elongated but not broken insofar as could be determined. These two joints were selected as extreme cases of straight and irregular cracking over the submerged parting strips used in creating this type of joint. The fractures that formed over these strips were, in general, meandering in character and were much wider than those that developed in the regular sections containing comparable percentages of continuously bonded reinforcement. Under the influence of traffic and exposure the edges of the fractures have raveled and spalled to a considerable width, creating a rather unsightly surface condition. This condition developed most rapidly during the first 2 or 3 years of service. Subsequently the deterioration in surface condition has been gradual.

The condition of the weakened-plane joints of the surface-groove type was excellent, initially, and continued to remain so except where there has been failure of the reinforcing steel. The present appearance of a typical joint is shown in figure 24. Very little maintenance has been required at these joints, so long as the steel remained structurally sound, since the comparatively small length changes of the 10-foot units are conducive to well-sealed conditions.

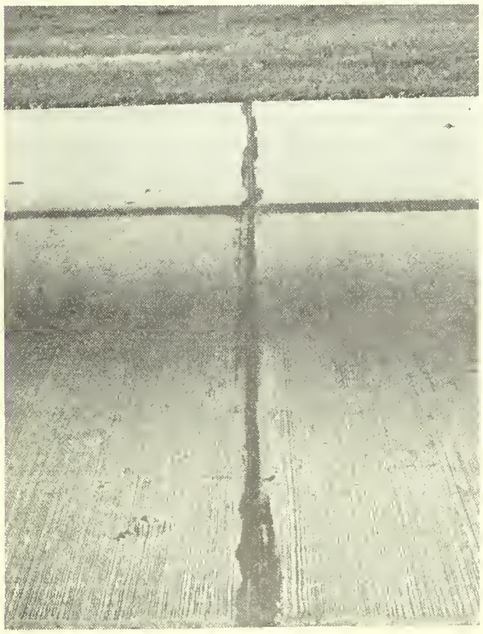

Figure 24.—Present condition of a surface-type, weakened-plane joint, typical of those at which the continuous reinforcement is structurally sound.

Progressive Changes at Joints

Daily, annual, and progressive changes in the widths of the joints of the four special sections were measured during the same periods as those of the regular sections. In figure 25 are shown the annual and progressive changes in the widths of the joints, plotted with respect to base measurements taken during the first winter after construction. The width changes of the expansion joints are, in reality, length changes of the sections as determined by measurement to fixed reference points located at their ends. Attention is called to the differences in the vertical scales used for the width changes of the expansion and weakened-plane warping joints, this being necessary because of the relatively small magnitude of the width changes of the latter. The lengths of the stippled bars indicate changes in width that occurred at the joints during an annual cycle. These changes should be nearly maximum for such a cycle since an attempt was made to obtain the measurements during the hottest and coldest periods of the year. All measurements in a given section were discontinued as soon as the first failure in the reinforcing steel was noted.

It is apparent in figure 25 that, in spite of the continuity of the reinforcing steel throughout the length of a section, the expansion joints closed and the weakened-plane joints opened progressively with time. These progressive changes are in the same sense as those observed in plain concrete pavements built with expansion joints and closely spaced weakened-plane contraction joints. It has been observed, also, that the progressive clos-

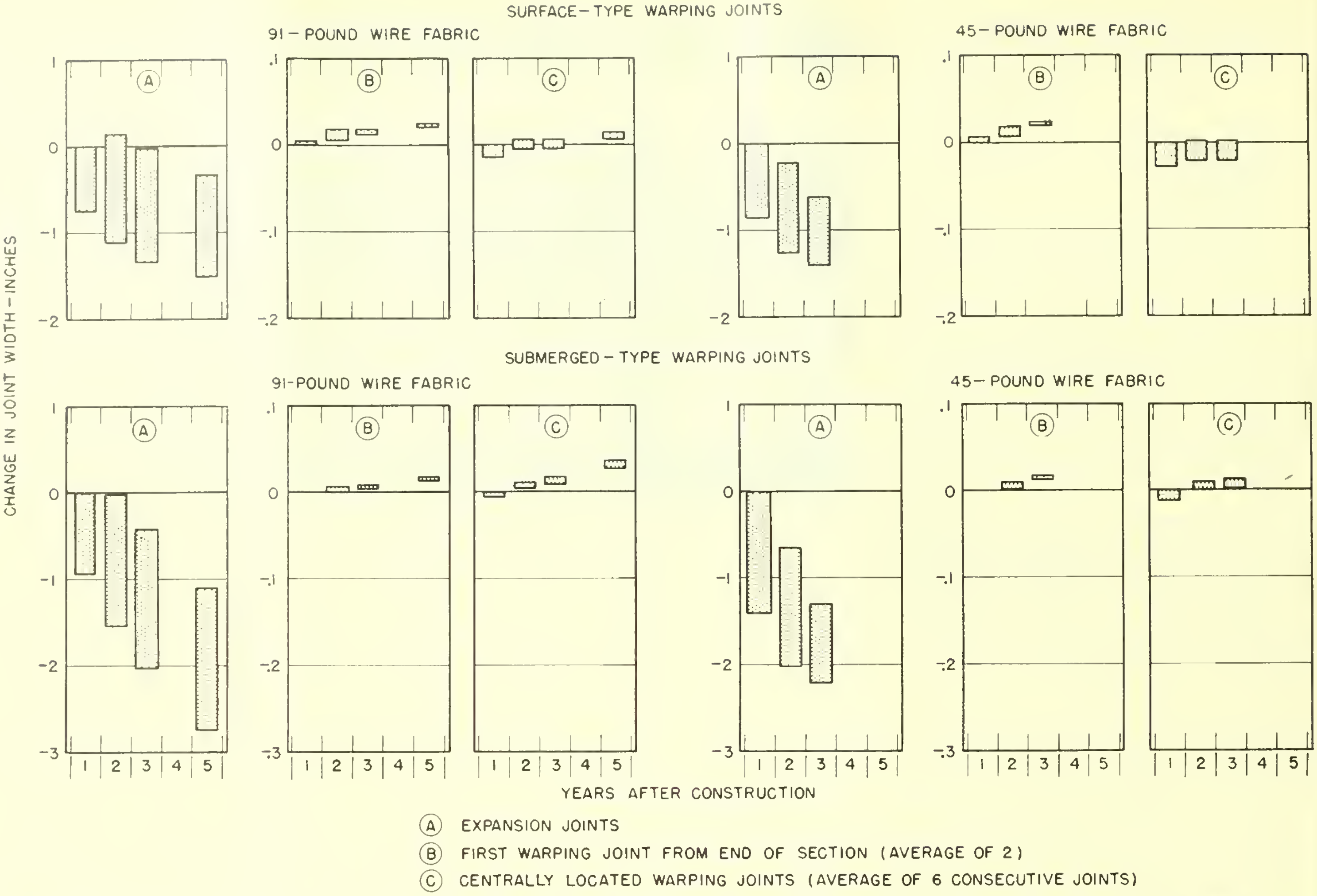

Figure 25.—Annual and progressive joint width changes of the 500-foot sections containing warping joints at 10-foot intervals: Values above the zero line denote opening; those below, closing, of the joint with respect to the base readings of December 1938.

ure of expansion joints at the end of 3 years has been less in sections reinforced with the 91-pound wire fabric than in those containing the 45-pound wire fabric; and less in sections with the surface-type joints than in those with the submerged type. This latter observation implies that extraneous material infiltrates more readily into the submerged-type joints, indicating that joints of this type are more difficult to seal. The behavior of the weakened-plane warping joints is somewhat erratic and, other than the fact that a progressive opening has developed in all cases, clear-cut trends are not apparent.

As remarked before, one of the purposes of the longitudinal steel in these 500-foot special sections was to hold the slab units of the sections together, as much as possible, during contraction periods. In this respect it is clearly shown in figure 25 that the reinforcing steel, especially the heavier fabric, exercises considerable control over the behavior of the sections during such periods. Measurements of the over-all section-length changes at the ends of the two sections reinforced with the 91-pound wire fabric showed 1.09 and 1.13 inches, respectively, for a mean pavement temperature drop of 77° F. that occurred between midsummer and midwinter of the second annual contraction period. This is shown in figure 25 as the difference between the lower end of the second bar and the upper end of the third bar in each of the four graphs marked *A*. These values represent approximately 77 percent of the annual contractive change of a section of equal length, but containing heavier, continuously bonded reinforcement such as was used in the regular sections. For the same temperature change the length changes of the two 500-foot special sections reinforced with the 45-pound wire fabric were 0.67 and 0.71 inch, respectively, or about 48 percent of the length changes of the comparable sections of the regular group. A similar comparison of the daily contractive length changes of the special sections with those of the section containing the continuously bonded steel results in values of approximately 54 and 37 percent, respectively, for the 91- and 45-pound wire fabric.

These comparisons show that the pattern of movement of the ends of the 500-foot special sections, although of less amplitude, is similar to that observed in the regular sections containing continuously bonded steel. It is apparent that during periods of contraction the heavier of the two weights of reinforcement in the special sections was more effective in holding together the individual slab units; and that both weights of reinforcement were more effective during annual periods than during daily periods.

It is of considerable interest that the small amount of longitudinal steel in the sections reinforced with the 45-pound wire fabric was able to cause contraction of the entire 500-foot section without steel failure. By any reasonable assumptions this would indicate that, when such contraction occurred, the coefficient of subgrade resistance was very much lower than the value of 1.5 assumed when the regular sections were designed. It appears also that the coefficient of resistance is smaller for the slow annual changes in length than it is for the more rapid daily changes, this being in agreement with data obtained on the regular sections.

Part 5.—THE OCCURRENCE OF PUMPING

It is generally conceded that three factors are necessary for the development of pumping at joints or cracks in concrete pavements: (1) frequent repetition of heavy axle loads and accompanying large vertical movements of slab edges; (2) fine-grained subgrade soils; and (3) free water under the pavement slab. Since the experimental sections were constructed as a part of a heavily traveled route, one of the factors, repetition of heavy axle loads, is always present. Moreover, the pavement was placed on a fine-grained subgrade soil that would be considered conducive

Table 8.—*Subgrade soil data*

	Silt	Clay	Liquid limit	Plasticity index	Moisture content		
					0–3 inches below surface	3–12 inches below surface	12–24 inches below surface
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Maximum	65	[1] 26	52	26	22.6	21.0	27.5
Minimum	20	7	19	4	6.1	8.9	8.1
Average	48	17	33	12	12.8	15.5	17.1

[1] This maximum percentage was exceeded in two instances; however, these cases were not considered as representative of the entire project.

to pumping. The soil analysis, shown in table 8, was based on samples taken from the finished subgrade at intervals of approximately 500 feet. Referring to the average values of the table, it is observed that the combined amount of silt and clay was 65 percent of the total.

Early in the life of the pavement, pumping began to appear at some of the bridge-type joints which were used to create the wider separations between the longer and consequently more heavily reinforced sections. These joints had no medium for load transfer except the steel cover plate. After 10 years most joints of this type were pumping and it has been noted that this action in some instances has resulted in serious faulting with transverse cracking of the forward and sometimes the approach slab. Of special interest is the fact that, in spite of the faulting, the heavy reinforcement has thus far held closed all cracks in the pavement areas adjacent to these joints. After 7 or 8 years pumping was observed at some of the conventional dowel joints which separate the shorter sections. To date, however, the action at these joints has been so slight that faulting is negligible and fracturing of the slabs has not occurred.

At the end of 10 years the performance survey showed that, with two exceptions, the only evidence of pumping in the entire pavement was in the vicinity of the transverse joints. One of the exceptions was the development of pumping at two of the cracks that formed in the sections containing the 32-pound wire fabric. As stated before, the reinforcing steel ruptured at several cracks in these sections, allowing wide separations. Pumping appeared shortly after the reinforcement failed.

The other exception was the appearance of pumping at one point along the edge of the heavily traveled lane some distance from the end of one of the most heavily reinforced sections. This condition was observed during the survey at the age of 10 years. Mud was not being ejected through any of the cracks, but was appearing at the shoulder and the pavement edge. The cracks in the immediate vicinity were seemingly as tightly closed and as nearly watertight as any in the section. For this reason, it is believed that water reached the subgrade by some other channel, along the pavement edge or possibly through the longitudinal joint. At three consecutive cracks in the immediate vicinity of the pumping, where the crack interval is about 2.5 feet, the edges of the cracks on the pavement surface are raveled or chipped to a more pronounced extent than at the other cracks in the section, indicating that the segments of pavement have been deflected considerably by heavy loads in spite of the presence of heavy longitudinal steel. After a few more years of service this condition may reach a point where some form of maintenance will be necessary.

Pumping Absent at Cracks

The complete absence of pumping at the vast number of transverse cracks in these sections, on a pumping type of soil, is evidence of the effectiveness of the reinforcing steel in holding tightly together the segments of the sections. Closed cracks not only minimize the leakage of free water to the subgrade but, by transferring load, minimize slab deflections as well.

Periodic observations of the weakened-plane warping joints of the special sections have shown that pumping developed at 10 of the 11 surface-type joints at which the wire-fabric reinforcement failed. In all 10 cases the action of pumping began shortly after steel failures were noted. Conversely, pumping did not appear as long as the wire fabric crossing the warping joints remained structurally sound; or at any of the seven submerged-type joints at which the reinforcing steel failed.

The preceding observations indicate that the entrance of free water to the subgrade and large slab deflections have activated pumping. Relatively wide separations (up to three-eighths inch) developed at all joints at which the reinforcement failed, thus reducing or destroying the effectiveness of aggregate interlock, and impairing the sealing of the surface-type joints. Of particular interest is the complete absence of pumping at the submerged-type joints, even those at which the wire fabric failed. When these joints were installed a copper seal which enveloped the bottom parting strip was incorporated in the design. Apparently these seals are still functioning as planned, despite the wide separation that has developed at the joints where the longitudinal steel has failed.

The effect of repetition of heavy axle loads on the development of pumping is clearly revealed in this investigation. As mentioned previously, the experimental pavement is one-half of a divided highway, with the result that the right-hand lane carries a greater number of heavy vehicles. At the end of 10 years the performance survey disclosed only one case of pumping in the left-hand or passing lane of the pavement, in contrast to the condition in the right-hand lane as just described.

Part 6.—PAVEMENT SMOOTHNESS

The common goal of all pavement design is a continued smooth riding surface, economics being, of course, a limiting factor. To evaluate the riding quality of the experimental sections, an instrument for indicating the relative roughness of road surfaces was used. With this device, which was developed some years ago by the Bureau of Public Roads,[5] it is possible to compare the surface roughness of the various sections by means of a roughness index, expressed in inches per mile of pavement. Low index values, of course, represent smooth pavement.

The basic set of data indicating relative values of surface roughness of these experimental sections was obtained in August of 1940, less than 2 years after construction. At that time roughness indices were determined only for the sections in the heavily traveled or right-hand lane of the pavement, it being presumed that, early in the life of the pavement, the surface of both lanes would be equally smooth. A second set of data was obtained in August of 1949, these data including both the heavily traveled lane and the passing lane so that the effect of traffic on surface roughness could be ascertained. Eliminated from these latter data was the localized condition of roughness found at locations where a pair of bridge-type joints were spaced 10 feet apart.

Average values of the surface roughness of short, intermediate-length, and long sections are given in table 9, the sections being grouped according to the range in lengths designated earlier in the report. All values were obtained with the single wheel of the roughness measuring vehicle traversing approximately a midlane path.

It is well to point out that, as a result of experience gained in using this equipment over many hundreds of miles of pavements of all types, it has been found that pavements with indices of the order of 80 to 120 have surfaces that would be classed as smooth riding.

The data of table 9 show that initially the pavement as a whole was very smooth indeed, indicating that the construction and, particularly, the finishing were unusually good. The fact that little difference was observed in the roughness indices of the three groups of sections suggests that, with proper care

Table 9.—*Roughness indices classified by length of sections*

Range in section length	Roughness index, in units per mile		
	1940 survey, right lane	1949 survey	
		Right lane	Left lane
Feet			
0–120	89	131	129
120–470	86	130	124
470–1,310	90	126	124

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[5] *Standardizable equipment for evaluating road surface roughness*, by J. A. Buchanan and A. L. Catudal; PUBLIC ROADS, vol. 21, No. 12, Feb. 1941.

During installation and finishing, the spacing of expansion joints need not affect the initial riding quality of concrete pavements.

A comparison of the roughness indices (table 9) determined in 1949 with those of 1940 indicates a marked increase in the surface roughness of all three groups of sections, the percentage increase in the units per mile being 47, 51, and 40, respectively, for the short, intermediate-length, and long sections of the heavily traveled lane. Even with this large percentage increase, the data indicate that the surface of the regular sections after 10 years of service is no rougher than some new pavements as constructed.

Comparison of Lanes

Only a slight tendency is noted, however, for the surface of the heavily traveled lane to become rougher, with time, than that of the passing lane. Also, in both lanes there is a slight but only a slight tendency for the pavement of the group of long sections to be smoother than that of the short sections, which contain relatively few cracks. It is apparent from these observations that, to date, traffic has had little effect on the increase in surface roughness; and that the many cracks that formed in the long sections have not affected the riding quality of the pavement.

In connection with the observed increase in pavement roughness between 1940 and 1949, it will be recalled that figure 1 shows examples of observed changes in pavement elevation, changes that developed principally through heaving and settlement of the subgrade and that undoubtedly account for part, at least, of the increase in surface roughness.

In table 10 are given the roughness indices for sections containing the various percentages of longitudinal steel for each of the three types of reinforcement. These data, although showing no particular trends for the factors involved, do show that a narrow range in the roughness indices of the various sections existed initially and still exists. This implies that all of the sections have remained structurally intact.

Table 10.—*Roughness indices classified by type and percentage of longitudinal steel*

Type of reinforcement and percentage of longitudinal steel in section	Roughness index, in units per mile		
	1940 survey, right lane	1949 survey	
		Right lane	Left lane
Percent			
Rail-steel bars (deformed):			
1.82	99	124	127
1.02	85	123	121
0.45	85	127	124
0.26	85	128	121
0.11	84	130	122
Billet-steel bars (deformed):			
1.82	85	128	123
1.02	78	130	123
0.45	90	130	121
0.26	88	125	128
0.11	91	135	129
Wire fabric (cold-drawn wires):			
0.42	90	137	129
0.28	90	133	124
0.24	89	131	127
0.17	89	137	139
0.11	84	120	124
0.07	98	134	126
Average	88	129	125

Roughness indices of the four 500-foot special sections were obtained at the same time as those of the regular sections. These data, given in table 11, are listed in accordance with the distinguishing features of the special sections.

At the time of the 1940 roughness survey the surfaces of the special sections were as smooth as those of the regular sections, the average index for the two types of pavement being 89 and 88 respectively. This indicates that the surface-type, weakened-plane joints were finished with great care and that the rather wide and meandering cracks which formed above the parting strips of the submerged-type joints did not impair the riding quality of the pavement at that time.

Table 11.—*Roughness indices of the four 500-foot special sections* [1]

Type of joint and weight of reinforcement	Shear bars	Roughness index, in units per mile		
		1940 survey, right lane	1949 survey	
			Right lane	Left lane
Lb. per 100 sq. ft.				
Submerged type:				
91	Yes	100	134	137
91	No	90	134	127
45	Yes	90	141	148
45	No	90	134	148
Surface type:				
91	Yes	79	141	116
91	No	90	120	169
45	Yes	79	148	127
45	No	95	218	137

[1] Each roughness index is based on 250 feet of pavement.

In 1949 the surfaces of the special sections were, in general, somewhat rougher than those of the regular sections. This greater increase in roughness is probably the result of the conditions at some of the joints that were described earlier, conditions that were not present at the time of the 1940 measurements. An example is the large increase in the roughness index of the half of the section with surface-type joints and containing the 45-pound fabric and no shear bars. It was in this half of the section that pumping developed at the joints where the reinforcing steel failed. This action has resulted in some tilting of the 10-foot slab units and a consequent faulting at the joints until, at present, this particular part of the pavement is quite rough.

New Publications

The *Highway Capacity Manual*, by the Committee on Highway Capacity, Department of Traffic and Operations, Highway Research Board, has now been published in book form by the Bureau of Public Roads and is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 65 cents.

The report, concerning the effectiveness of various highway facilities in the service of traffic and involving the many elements of highway design, vehicle and driver performance, and traffic control, was originally published in PUBLIC ROADS, vol. 25, Nos. 10 and 11 (October and December 1949) under the title "Highway Capacity: Practical Applications of Research." The 147-page reprint, in convenient 6- by 9-inch book size, will be a valuable addition to the library of every design and traffic engineer.

The manual encompasses both rural and urban facilities. Its seven major sections provide extensive discussion of maximum observed traffic volumes, fundamentals of highway capacity, roadway capacities for uninterrupted flow, signalized intersections, weaving sections and unsignalized cross movements, ramps and their terminals, and the relation of hourly capacities to annual average volumes and peak flows.

The study is based on a great mass of field observations, made available by the cooperative efforts of the Bureau of Public Roads, the Highway Research Board Committee on Highway Capacity, and many State, county, and city engineers, intensively applied in many places and for a number of years. From the vast grist of basic data, painstakingly analyzed, has been evolved this practical guide to rational methods for the determination of highway capacity, essential in the sound economic and functional design of new highways and in the adaptation to present or future needs of the many existing roads and streets which must continue in use for extended periods of time.

***Highway Statistics, 1948*, is also now available from the Superintendent of Documents. The fourth in an annual series, this publication presents statistical information of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, State highway finance, and highway mileage for the year 1948. The summary bulletin reporting similar information over periods of 20 to 50 years, up to 1945, is a valuable adjunct to the annual series. These publications are sold by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at the following prices:**

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF FEBRUARY 28, 1950

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$15,347	$13,966	$7,142	392.4	$7,143	$3,518	141.7	$10,731	$6,160	225.6	$31,840	$16,820	759.7
Arizona	2,064	6,193	4,324	165.8	1,451	1,011	36.0	5,019	3,425	55.6	12,663	8,760	257.4
Arkansas	6,870	10,224	5,635	308.0	6,662	3,232	158.5	10,863	5,488	222.8	27,749	14,355	689.3
California	7,009	47,449	17,969	287.6	3,937	1,977	33.6	36,207	18,596	168.9	87,593	38,542	490.1
Colorado	7,966	1,378	761	23.7	5,398	2,886	88.8	11,555	6,838	229.3	18,331	10,485	341.8
Connecticut	5,362	8,198	3,733	17.8	2,132	1,416	3.2	5,517	2,986	6.7	15,847	8,135	27.7
Delaware	3,040	2,251	1,123	44.2	2,072	1,031	33.0	2,008	954	23.0	6,331	3,108	100.2
Florida	10,038	11,475	6,225	192.3	4,094	2,270	96.8	8,454	3,983	216.3	24,023	12,478	505.4
Georgia	7,298	18,974	9,943	672.1	4,766	2,386	188.9	41,337	19,541	604.8	65,077	31,870	1,465.8
Idaho	5,885	10,475	6,601	434.3	1,198	686	42.7	4,363	2,675	92.1	16,036	9,962	569.1
Illinois	27,464	46,270	24,068	439.0	11,367	6,163	122.9	42,377	20,314	295.8	100,014	50,545	857.7
Indiana	18,137	20,611	10,235	104.4	7,114	3,426	34.7	9,948	5,781	21.7	37,673	19,442	160.8
Iowa	7,541	15,119	7,085	858.9	3,304	1,596	138.7	14,701	7,481	397.2	33,124	16,162	1,394.8
Kansas	7,324	11,081	5,529	1,091.7	7,536	3,750	486.1	13,283	6,834	857.3	31,900	16,113	2,435.1
Kentucky	3,006	22,907	11,361	417.1	5,020	2,522	105.7	12,170	5,955	179.5	40.097	19,838	702.3
Louisiana	8,691	23,028	11,172	239.3	6,772	3,768	51.6	14,186	6,308	136.4	43,986	21,248	427.3
Maine	4,261	7,285	3,961	102.3	219	115	4.2	5,731	2,850	77.7	13,235	6,926	184.2
Maryland	3,304	8,266	3,970	54.9	409	204	6.1	15,752	7,671	51.7	24,427	11,845	112.7
Massachusetts	1,258	28,300	13,358	41.1	18,557	8,616	20.8	37,069	17,722	22.7	83,926	39,696	84.6
Michigan	11,954	18,223	9,420	511.0	5,748	2,886	102.1	37,581	15,043	165.8	61,552	27,349	778.9
Minnesota	6,617	15,453	8,266	699.2	7,863	4,076	192.1	12,865	6,572	142.9	36,181	18,914	1,034.2
Mississippi	15,100	1,646	822	61.9	250	123	6.4	9,568	5,016	208.9	11.464	5,961	277.2
Missouri	7,992	37,291	19,394	1,001.7	14,409	5,822	250.8	23,331	12,265	480.9	75,031	37,481	1,733.4
Montana	12,436	10,503	5,222	313.2	2,406	1,427	100.9	9,968	6,268	219.4	22,877	12,917	633.5
Nebraska	8,231	18,139	9,400	700.8	5,317	2,967	78.2	8,946	5,171	186.1	32,402	17,538	965.1
Nevada	5,021	3,809	3,152	94.8	691	568	24.5	3,512	2,861	100.4	8,012	6,581	219.7
New Hampshire	2,423	6,938	3,435	66.8	646	304	2.0	1,532	741	10.3	9,116	4,480	79.1
New Jersey	3,760	11,579	5,690	20.4	3,106	1,553	1.4	18,637	8,881	29.8	33,322	16,124	51.6
New Mexico	2,333	11,589	7,402	428.1	3,145	2,028	73.6	7,037	4,583	211.6	21,771	14.013	713.3
New York	53,538	57,711	30,610	241.0	12,831	7,585	25.7	81,688	40,303	152.8	152,230	78,498	419.5
North Carolina	12,527	12,097	5,996	278.3	5,590	2,595	132.9	17,340	8,457	375.3	35.027	17,048	786.5
North Dakota	4,625	11,203	5,772	1,372.3	4,093	2,124	219.2	6,354	3,354	465.5	21,650	11,250	2,057.0
Ohio	9,214	74,131	35,945	321.2	13,612	6,460	94.0	28,326	14,015	116.7	116,069	56,420	531.9
Oklahoma	1,160	35,248	18,339	767.3	6,697	3,202	181.8	10,251	5,304	268.3	52,196	26,845	1,217.4
Oregon	2,276	9,418	5,504	164.7	2,706	1,579	88.8	10,176	5,428	86.7	22,300	12,511	340.2
Pennsylvania	16,821	33,296	16,648	93.8	9,277	4,858	37.7	69,398	33,972	151.8	111,971	55,478	283.3
Rhode Island	1,627	7,271	3,653	37.2	7,000	4,023	1.1	4,031	1,576	4.2	18,302	9,252	42.5
South Carolina	5,390	8,046	4,057	362.6	1,420	775	101.7	11,023	5,544	393.5	20,489	10,376	857.8
South Dakota	2,820	14,690	8,538	1,417.4	2,534	1,713	273.8	5,183	3,173	330.9	22,407	13,424	2,022.1
Tennessee	7,040	12,232	6,146	272.6	9,298	4,444	168.5	14,888	6,858	190.8	36,418	17,448	631.9
Texas	15,852	7,539	3,945	639.4	13,580	6,640	652.5	62,154	28,863	1,516.5	83,273	39.448	2,808.4
Utah	5,281	2,343	1,732	88.0	1,502	1,121	40.0	5,320	3,863	125.9	9,165	6,716	253.9
Vermont	1,165	4,387	2,377	44.3	115	56	.1	4,271	1,963	40.6	8,773	4,396	85.0
Virginia	14,089	14,429	7,248	394.6	4,597	2,235	182.0	12,620	6,257	178.6	31,646	15,740	755.2
Washington	3,520	15,657	6,902	199.7	6,550	2,724	74.4	13,263	6,117	64.9	35.470	15,743	339.0
West Virginia	2,877	17,212	7,216	196.7	4,067	2,139	38.9	6,898	3,403	33.7	28,177	12,758	269.3
Wisconsin	13,006	24,137	12,421	479.5	1,566	741	46.7	12,112	6,029	286.5	37,815	19,191	812.7
Wyoming	1,310	6,065	3,778	220.5	1,636	1,120	25.4	5,782	3,746	210.5	13,483	8,644	456.4
Hawaii	2,042	6,919	3,130	32.7	4,049	1,871	8.9	4,289	2,208	15.4	15,257	7,209	57.0
District of Columbia	2,823	3,370	1,897	.5				11,091	5,510	1.9	14,461	7,407	2.4
Puerto Rico	5,420	8,868	4,010	33.9	1,479	635	7.3	9,797	3,610	48.0	20,144	8,255	89.2
TOTAL	412,155	834,889	422,262	17,443.0	256,931	130,967	5,027.4	840,503	418,516	10,700.2	1,932,323	971,745	33,170.6

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Dual-purpose commercial parking structures, with street-level shops, help solve the down-town parking problem

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BUREAU OF PUBLIC ROADS

U. S. DEPARTMENT OF COMMERCE

E. A. STROMBERG, Editor

Evidence of the parking problem: cars double-parked on the street waiting for space in a lot that is already full.

Some Travel and Parking Habits Observed From Parking Studies

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH, BUREAU OF PUBLIC ROADS

Reported by S. T. HITCHCOCK, Assistant Branch Chief, and R. H. BURRAGE, Highway Engineer

Riding past the "full-up" parking lots and making endless circuits of the block in search of a vacant curb space, every motorist has become personally aware of the critical downtown parking problem. To resolve this problem, detailed information on all its ramifications is first needed; and in 46 cities, ranging in population from under 6,000 to more than a million, comprehensive parking studies of the central business districts have been undertaken or completed. From these studies, conducted according to a common basic pattern, some interesting observations on parking habits have been drawn.

It has been found that the number of available parking spaces and the number of vehicles parked, in proportion to population, decrease rapidly as the size of the city increases—there being seven times as many spaces and thirteen times as many parked vehicles, per 1,000 population, in the small cities as in the large cities. The central business districts of the small cities generate ten times as much traffic, in the peak half-hour, as those of the large cities, when proportioned to population. The percentage of traffic passing through the central business district during the day as a whole is about the same for all cities, but during peak periods 91 percent of the traffic in large cities does not stop to park. In the concentrated core of the central business districts, space-hour demands for parking exceed the supply by ratios ranging from 1.34 in small cities to 4.67 in the large cities.

FULL economic benefits of urban arterial and expressway street and highway developments cannot be realized if there is no place to park the motor vehicle at the terminus of the trip. The planning of adequate terminal facilities for the parking of cars or the loading and unloading of trucks is consequently an integral part of any plan for urban highway transportation improvements.

In many cities the parking problem in the downtown area was serious before the war, and with the postwar increase in motor-vehicle usage it has reached alarming proportions. In an effort to evaluate the parking load under present conditions—a load which would most certainly increase with the improvement of arterial streets and the construction of expressways leading to and from the downtown area—many cities have made studies of traffic volumes and travel habits as they related to trip destinations and parking in the downtown area.

If a parking study is to be of value in planning for the development of off-street parking lots or garages, or in determining reasonable restrictions for curb usage, certain basic information is necessary:

1. Location and existing supply of parking spaces.
2. Usage of existing parking spaces.
3. Location and extent of existing demand.
4. Cordon count of inbound and outbound traffic to determine total parking load.
5. Influence of large traffic attractions on parking demand.
6. Parking habits.
7. Adequacy of existing laws and ordinances.
8. Limitations of existing administrative responsibility.
9. Financing methods.

The metropolitan area travel habit studies, which have proved extremely useful in estimating probable desire lines of travel and traffic usage of selected route locations, provide some information on parking practices.[1] The size of the sample, however—usually 5 or 10 percent—proved to be inadequate for the estimating of trip destination volumes for the relatively small areas such as would be usable for determination of location and size of parking facilities.

One specific method of making a parking study, commonly referred to as "cruising," is based on the inspection methods used by police for enforcement of parking regulations. License numbers of parked vehicles are recorded at periodic intervals either from a moving car or by a recorder on foot. This method indicates the usage of space, as to location and length of time parked, within limits of the length of time between recordings. It sacrifices all information on the short-time parker, and does not permit obtaining any information on trip purpose and destination, which is desirable for determining the proper location of parking facilities.

Another method, involving the distribution of return-mail questionnaire cards, has also been used in some spot situations. For a comprehensive analysis of the entire downtown area this is considered to be unsuitable because of the uncertain quantity of returns (seldom exceeding 30 percent) and the lack of statistical control of those returned.

Because of the limitations of these approaches and the need for more specific information for purposes of location selection and capacity design a more comprehensive type of study has been developed,[2] in which the needed data are obtained by direct interview of parkers and by cordon count of all vehicles entering and leaving the area.

Summary of Observations

Analysis of data fully developed in 29 cities and partial information from 17 others, where comprehensive parking studies of the central business districts have been conducted in a uniform manner, reveals many pertinent and interesting trends in parking habits.

As would be expected, the areas of central business districts of cities, where the parking situation is most acute, are relatively small and directly proportional to population—ranging from 0.1 square mile in the small cities to 0.5 square mile in the largest. In relation to population, of course, this range is reversed: the small cities having 0.7 square mile per 100,000 population and the largest, 0.05. Similarly, the total number of parking spaces available ranges from 1,400 in the smallest cities to 10,200 in the largest, while per 1,000 population the spread is in the opposite direction, from 89 to 12.

The number of vehicles parked during the 8-hour business day varies from 7,100, or 446 per 1,000 population, in the small cities, to 30,000, or 34 per 1,000 population, in the large ones. In all cities the proportion of commercial vehicles parked is fairly constant at about 13 percent.

The parking problem is most acute, of course, in the core of the central business district. There, the space-hour demand ranges from 3,500 in the smallest cities to 28,600 in the largest cities, while the available supply is from 2,600 to 6,600. The ratio of demand to supply increases from 1.34 in small cities to 4.67 in large ones.

For various reasons, the central business districts of small cities are proportionately greater traffic generators than those of the large cities. The in-and-out traffic in the peak half-hour, per 1,000 population, ranges from 139 vehicles in the former to only 14 in the latter. The ratio of the peak-hour traffic to that in the average hour, however, is constant regardless of population, the peak-hour volumes being about one-third greater. The proportion of traffic passing through the central business districts during the business day varies but little for cities of different size, but traffic passing through in the peak half-hour ranges from 59 percent in the small cities to 91 percent in the large cities.

As a reflection of the different characteristics of small and large cities, it was found that 56 percent of the vehicles in the former and 28 percent in the latter parked less than 30 minutes; while 8 percent and 25 percent, respectively, parked 4 hours or more. Similarly, 70 percent of the parkers in small cities and 46 percent in the large cities walked less than a block to their destinations; while 9 percent and 30 percent, respectively, walked two blocks or more.

Regardless of the purpose of trip, the length of parking time increased in direct relation to the size of the city. The average time parked where "work" was the purpose ranged from 3 hours in small cities to over 5 hours in large cities, and similar spreads occurred for "shopping" and "business" purpose parking. Parking time for other purposes varied little, regardless of population.

The psychological advantage of parking meters is indicated by the fact that overtime parking, both in number of occurrences and in excess time parked, is considerably less at metered curbs than at unmetered but time-restricted curb spaces.

Table 1.—*Cities in which direct-interview type parking studies have been made*

City	Population [1]	Year of study	Data available
Decatur, Ind	5,861	1948	--
Seymour, Ind	8,620	1948	--
Wabash, Ind	9,653	1948	--
Columbus, Ind	11,738	1948	--
Albert Lea, Minn	12,200	1947	X
Frankfort, Ind	13,206	1948	X
Huntington, Ind	13,903	1948	X
Portsmouth, N. H	14,821	1946	X
Stevens Point, Wis	15,777	1947	X
Walla Walla, Wash	18,109	1946	--
Meadville, Pa	18,919	1948	X
Anderson, S. C	19,424	1947	X
Lake Charles, La	21,207	1947	--
Boise, Idaho	26,130	1948	--
Alexandria, La	27,066	1947	X
Monroe, La	28,309	1947	X
Easton, Pa	33,589	1948	X
Kokomo, Ind	33,795	1948	--
Anderson, Ind	41,572	1948	X
Lynchburg, Va	44,541	1948	--
Muncie, Ind	49,720	1948	--
Corpus Christi, Tex	70,700	1947	X
Pawtucket, R. I	75,797	1945	X
Charlotte, N. C	113,000	1947	X
Wichita, Kans	127,300	1947	X
Spokane, Wash	141,400	1947	X
Knoxville, Tenn	151,800	1946	X
Harrisburg, Pa	173,400	1946	X
Reading, Pa	175,300	1947	X
Chattanooga, Tenn	193,200	1947	X
Jacksonville, Fla	195,600	1947	--
Nashville, Tenn	241,800	1946	X
Honolulu, T. H	245,000	1947	--
Richmond, Va	245,700	1948	X
Omaha, Nebr	287,700	1948	--
New Haven, Conn	308,200	1946	X
Allentown-Bethlehem, Pa	325,142	1948	--
Toledo, Ohio	341,700	1947	X
Denver, Colo	384,400	1945	X
Portland, Oreg	406,400	1946	--
Atlanta, Ga	442,300	1945	X
Seattle, Wash	452,600	1946	X
Providence, R. I	711,500	1945	X
Baltimore, Md	1,046,700	1946	X
Cleveland, Ohio	1,215,000	1948	--

[1] U S. census of population, 1940.

The Comprehensive Parking Study

The purposes of a comprehensive parking study have been indicated briefly. Some elaboration is desirable, however, concerning the basic phase of the study operation, that of the interviews with the parkers. The study is as a rule confined to the central business district (or to any other area where parking is a problem), around which the cordon count is made. A direct interview is made with the driver of every vehicle parked in the area. The interviews usually are confined to the hours of the business day, 8 or 10 a. m. to 6 p. m., and hence include the peak of parking and traffic accumulation.

Table 2.—*Area and population relations of central business districts*

Population group	Number of cities	Average population in metropolitan area	Average area of the central business district		
			Square miles		Number of blocks [1]
			Total	Per 100,000 population	
Under 25,000	10	15,400	0.11	0.69	23
25,000–50,000	4	32,600	.13	.41	38
50,000–100,000	2	66,550	.22	.27	36
100,000–250,000	10	176,200	.44	.26	76
250,000–500,000	6	280,700	.46	.12	97
Over 500,000	2	879,000	.54	.05	134

[1] Block dimensions vary from 150 feet to 600 feet.

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[1] *Traffic planning studies in American cities*, by J. T. Lynch; Public Roads, vol. 24, No. 6, Oct. 1945; and *Highway safety: Driver behavior—key to safe highway design*, by Thos. H. MacDonald; Public Roads, vol. 25, No. 7, March 1949.

[2] *Factual guide on parking for the smaller cities*, Public Roads Administration, Sept. 1947; and *Determining parking requirements by means of a study of parking habits*, by S. T. Hitchcock and T. E. Willier; Proceedings of the Highway Research Board, vol. 24, 1944, p. 255.

It would be possible to accomplish the interviewing in one day by using a very large force, but it is preferable to use a small group of trained interviewers and to spread the work over several weeks. This affords advantages from an operating standpoint and produces data more representative of an average day. The cordon count is similarly a composite sample, spread over the same period as the interviews.

The actual interview with each driver requires only about 30 seconds and includes queries to determine trip origin, purpose in making the stop, and the destination to which the parker is going to walk. Data on the location, the time of arrival and departure, the type of vehicle, and the type of parking are obtained by observation. The data permit a determination of accessory but valuable information on turn-over, violation of space and time regulation, and use (and abuse) of loading zones and other special-privilege facilities. The information on how long drivers do park, when coupled with their destinations, provides a reasonably accurate measure of where they would like to park and for how long, if space were available. This is the existing demand.

The inventory of existing parking space and the cordon-count data are summarized and tabulated by manual methods. The interview data, because of the number of items of information involved and the many interrelations desired, are particularly susceptible to machine tabulation. Resultant from these summarizations is the information on generators of parking demand and on the parking characteristics of the motorists. The study of the legal, administrative, and financial aspects of the problem is a staff responsibility, to be integrated with the findings and recommendations from the other phases of the studies.

With the knowledge that the basic volumetric data have been obtained with reasonable accuracy, specific locations and designs for additional facilities may be planned with assurance. The information on parking habits, when correlated with location, may be used to advantage in revising time restrictions, and in the location or relocation of parking meters. It is also basic in determining operating characteristics and capacity for design purposes. The data on truck destinations and parking-time durations will indicate the need for added loading zones or off-street loading space.

Trends Observed From Studies

The direct-interview type of parking study, initiated in 1945, has now been made in 46 cities throughout the country, ranging in population from 5,000 to over 1,000,000 (table 1). Data have been developed fully in 29 of these cities. Since the same procedures were used in each case, it has been possible for the first time to observe some relations of area characteristics, parking habits, travel habits, and traffic volumes.

The data reported in this article represent conditions in cities which have recognized the existence of a parking problem sufficiently acute to warrant an extensive study as a first step toward seeking a solution. In all probability the parking problem is just as serious in a great many other cities; and it may be a mark of greater initiative on the part of the cities that have undertaken these parking studies, rather than of greater need.

Table 3.—*Curb and off-street parking spaces available in the central business district*

Population group	Number of cities	Average number of parking spaces			Spaces per 1,000 population	
		Curb	Off-street	Total	Curb	Total
Under 25,000	9	933	485	1,418	60	89
25,000–50,000	4	1,287	742	2,029	41	65
50,000–100,000	2	1,688	2,401	4,089	23	57
100,000–250,000	9	2,867	3,886	6,751	17	42
250,000–500,000	6	2,961	8,132	11,093	7	28
Over 500,000	[1] 2	2,510	7,675	10,185	3	12

[1] Providence, a city of 253,500 population, has a metropolitan area population of 711,500 with several other fully developed but smaller independent business districts.

The data available from the 29 cities permit analysis in several directions. The summaries presented here are some of the more obvious relations developed in initial analyses. For those cities where the time periods studied were not identical, data were adjusted to a common 8-hour basis (10 a. m. to 6 p. m.).

The data reported are averages for cities in each of six population groups. As will be seen from the tables and graphs, the numerical values of the various elements of the parking problem, as averaged by population groups, show clearly established trends that may be of value to other cities where comprehensive studies have not been made and where complete data are not available.

It had already been suspected that some of the indicated trends or relations might exist, and for some readers personal experience may make some observations appear obvious; but they substantiate many points which heretofore have been largely a matter either of opinion or conjecture. Furthermore, the fact that these data and derivations fall into a pattern indicates that the basic approach to this research problem, or at least the procedural technique, is soundly conceived.

It must be admitted that data for the individual cities are somewhat scattered, in contrast to the uniformity of the population-group averages. Quite probably this incongruity can be accounted for partially by the small number of cities for which data were available; but much more important factors are the character of business and industry and of geographical location, which may differ greatly among cities of the same size. The fact that all of the data as averaged by population groups fall in straight-line or smoothly curved patterns is strongly indicative of their close relation to population.

The Central Business District

The central business district of a city is not a legal entity nor is it generally a clearly defined area. In establishing the limits of such a district for the purposes of a parking study, it was considered as the area where land occupancy is nearly 100 percent, where land

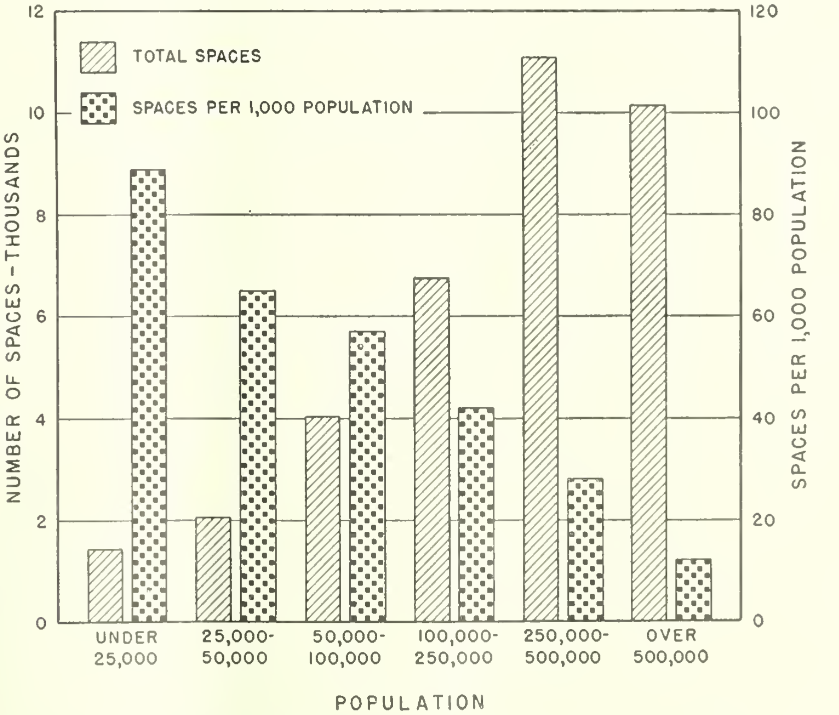

Figure 1.—Parking spaces available in the central business district.

Table 4.—*Number of vehicles parked in the central business district*

Population group	Number of cities	Average number of vehicles parked in 8 hours [1]		Maximum number accumulated [2]		Percentage commercial	Parking accumulation ratio [3]
		Total	Per 1,000 population	Total	Per 1,000 population		
Under 25,000	9	7,097	446	1,050	67	14	1.17
25,000-50,000	4	8,461	267	1,373	43	14	1.13
50,000-100,000	2	11,866	164	2,185	30	11	1.15
100,000-250,000	8	22,130	128	5,447	29	12	1.15
250,000-500,000	5	32,436	83	8,245	21	13	1.15
Over 500,000	2	29,957	34	9,564	11	13	1.11

[1] Total number parking during the period, including those parked at start of period. Adjusted, where necessary, to a common period 10 a. m. to 6 p. m.
[2] At any time during the 8-hour period.
[3] Ratio between the number of vehicles in parking spaces, legal or illegal, in the peak hour and in the average hour.

Table 5.—*Usage of space in the entire central business district, and the relation of demand and supply in the core area*

Population group	Central business district			Core area [1]			
	Number of cities	Present usage in space-hours		Number of cities	Space-hours		Ratio of demand to supply
		Average number	Per 1,000 population		Demand [2]	Supply	
Under 25,000	8	7,692	487	5	3,465	2,552	1.34
25,000-50,000	4	9,988	314	3	4,230	3,043	1.38
50,000-100,000	2	14,632	202	2	4,290	2,964	1.45
100,000-250,000	6	38,802	225	7	17,188	11,285	1.64
250,000-500,000	4	51,578	133	3	20,828	6,505	3.27
Over 500,000	2	65,846	74	2	28,590	6,649	4.67

[1] The core area is that portion of the central business district where land values are generally highest, and where in each block, of several contiguous blocks, the demand for parking space in each exceeds the supply.
[2] Demand for space in the core area, based on destinations of drivers who parked in the central business district.

use is principally business, where curb parking is congested, and to which transit lines converge. It is the area where are found the large stores and office buildings. It often includes some theaters and hotels. It may have a few apartments, but few or no single-unit residences and little manufacturing. Area and population data for the central business districts of a number of cities are shown in table 2.

The determination of the central business district was made locally in each city and, although different engineers established the limits, it is significant to note that the characteristic limits of the central business district have been generally recognized. The trend in size is perhaps to be expected, but confirmation of this trend lends assurance to further analyses in these cities.

In a city where studies have not been made, comparisons with the tabulated data should take into consideration whether the central business district constitutes a problem area greater or less than the average for a city of its population class.

Availability of Parking Space

It is to be expected, as shown in table 3 and figure 1, that as cities grow the supply of curb spaces for parking will decrease proportionally. Curbs are limited in physical extent and as the downtown area grows vertically more curb space is restricted for services in connection with adjacent buildings and for the movement of traffic. It should also be expected that off-street facilities would be developed in a compensating manner, but this apparently has not occurred. It will be noted that the cities of more than 250,000 population have less than one-half as many total parking spaces per capita as cities of less than 100,000 population.

The trends shown, even though expected, lend assurance to the soundness in determination of the limits of the central business district, to the use of the data for comparative purposes in cities where extensive studies have not been made, and to the procedures used in making the study.

Usage of Parking Space

As cities grow, the volume of parking both for the business day and for the peak hour increases. However, when these are expressed in volumes per 1,000 population, as shown in table 4, it is evident that parking usage in the central business districts is proportionately greater in the smaller cities than in the larger cities. This probably is due largely to the higher car ownership and more concentrated business in the smaller cities; and to the extent of mass transit, the presence of numerous suburban shopping areas, and the greater difficulty of finding parking space in the downtown sections, in the larger cities.

The proportion of commercial vehicles parking in the central business district apparently does not vary appreciably in cities of different size. These are the vehicles picking up and delivering goods in the downtown area.

Similarly there is not a great difference, among the various population groups, in the parking accumulation ratio—the ratio of the volume parked in the peak hour of parking and the volume parked in the average hour of the business day. This, perhaps, is traceable to the fact that studies have been made only in cities where a parking problem exists and where, therefore, the load is not far from peak condition all through the business day.

The data in table 4 represent only the volume of parking under present conditions and undoubtedly are not representative of what the trends might be if better traffic service and parking facilities were available.

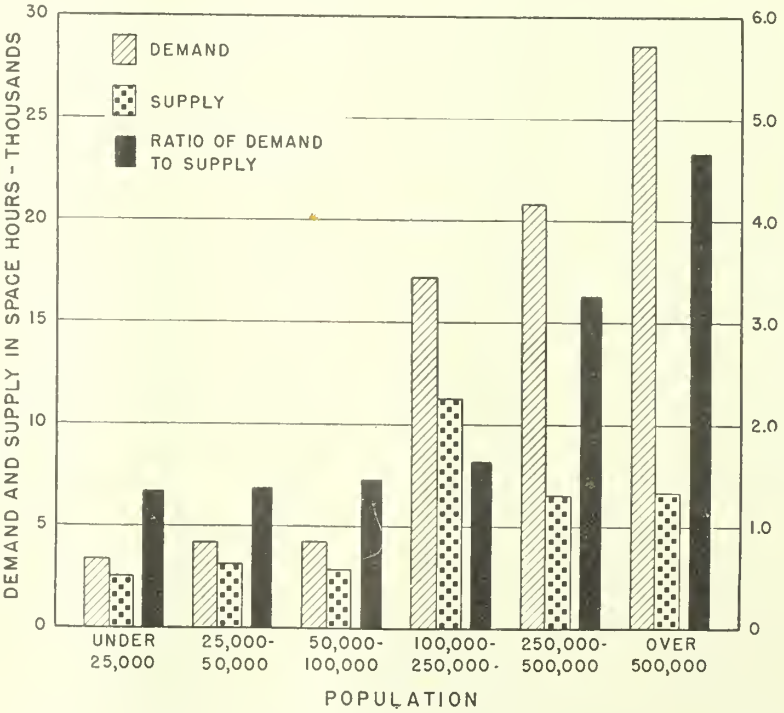

Figure 2.—Demand for and supply of parking space in the core area of the central business district, in terms of space hours.

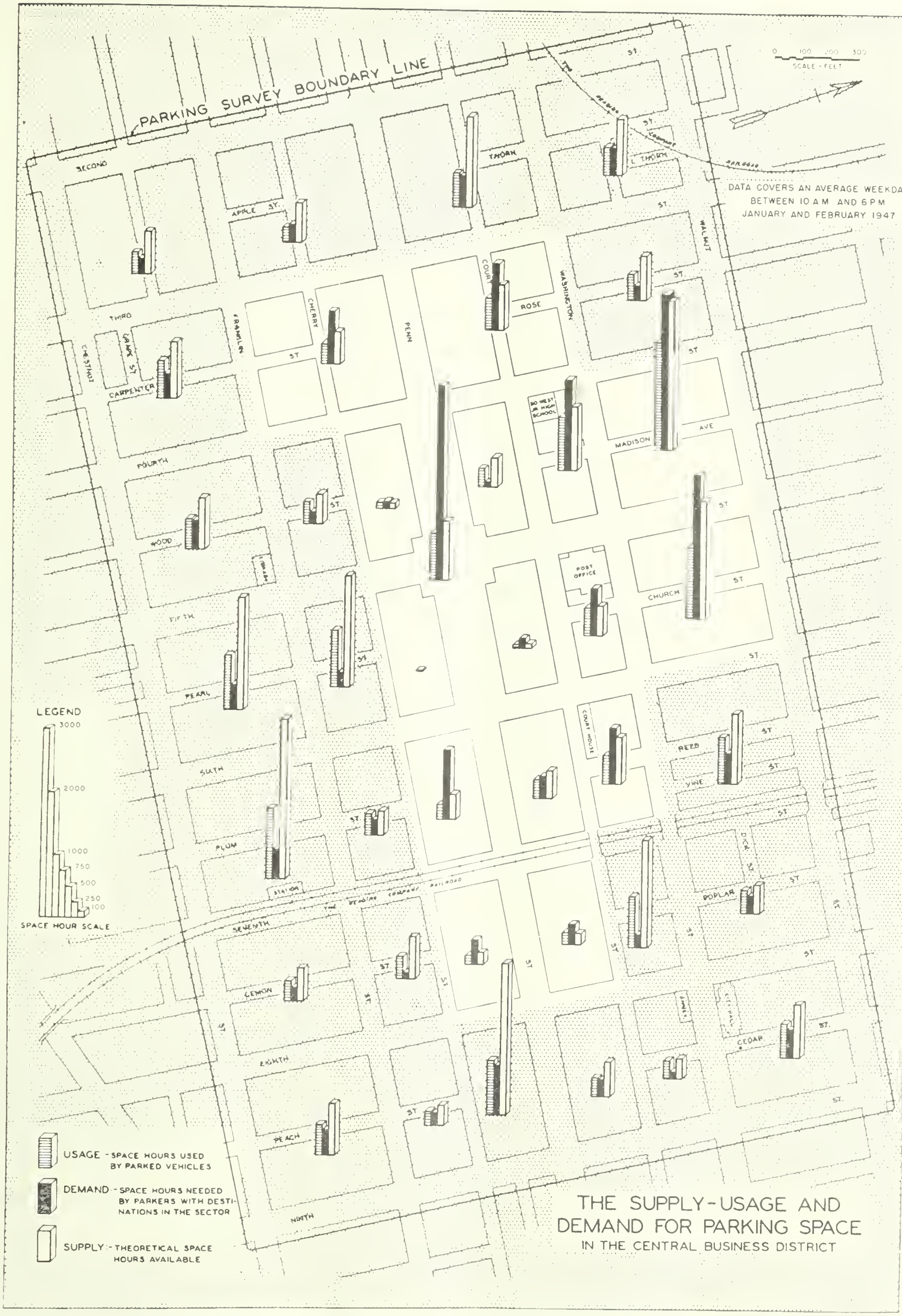
PARKING SURVEY BOUNDARY LINE
0 100 200 300
SCALE - FEET
DATA COVERS AN AVERAGE WEEKDAY BETWEEN 10 A M AND 6 P M JANUARY AND FEBRUARY 1947
SECOND
APPLE ST.
THORN
THORN
ST.
WALNUT
THIRD
GRAPE ST
CHESTNUT
CARPENTER
FRANKLIN
CHERRY
ST
PENN
COURT
ROSE
WASHINGTON
ST
BO WEST JR HIGH SCHOOL
MADISON
AVE
FOURTH
WOOD
LIBRARY
POST OFFICE
CHURCH
ST
FIFTH
PEARL
SIXTH
PLUM
STATION
COURT HOUSE
REED
VINE
DICK ST
POPLAR
THE READING COMPANY RAILROAD
SEVENTH
LEMON
EIGHTH
PEACH
NINTH
CITY HALL
CEDAR
LEGEND
3000
2000
1000
750
500
250
100
SPACE HOUR SCALE
USAGE - SPACE HOURS USED BY PARKED VEHICLES
DEMAND - SPACE HOURS NEEDED BY PARKERS WITH DESTINATIONS IN THE SECTOR
SUPPLY - THEORETICAL SPACE HOURS AVAILABLE
THE SUPPLY-USAGE AND DEMAND FOR PARKING SPACE
IN THE CENTRAL BUSINESS DISTRICT

Figure 3.—The needs for additional terminal facilities are pinpointed on this map of supply, usage, and demand for parking space in the central business district of a typical city of 175,000 population.

Parking Space Supply and Demand

The parking studies, made under existing conditions, provide information only for those drivers who park in the central business district. It does not include the potential demand of those who might park there under different conditions.

For those having destinations in the central business district the demand for parking space shows a definite increase with the size of the city, as indicated in table 5. On a per capita basis, however, the central business districts in the smaller cities are greater parking generators than those in the larger cities, for reasons previously stated.

For the central business district as a whole, the supply of spaces is usually found to be at least equal to the demand since, for study purposes, the limits are so selected as to include the parking of nearly all drivers bound for the district. Some of the central blocks in the district, however, create a demand exceeding the supply in the same blocks. Table 5 and figure 2 indicate that the volume of demand for spaces in this core area increases in the larger cities whereas the supply, although increasing in cities of medium size, drops off in the larger cities where high land values in the core tend to preclude the use of space for parking purposes.

Figure 3 illustrates the relation of parking-space supply, usage, and demand in a typical city of about 175,000 population. Scaled bars show, for each block, the parking space available, the usage of that space, and the demand expressed for parking space in that block. The use of space-hour units permits direct comparisons. The unshaded central portion of the map shows the core area of the central business district, and covers those blocks in which the demand for space exceeds the supply. The map thus graphically pinpoints areas deficient in parking space.

Traffic and Travel Habits

As cities increase in population, the total 8-hour inbound volume of traffic, the average hourly volume in and out, and the peak volume in and out of the central business district increase, as shown in table 6. But when the population of the city is considered, the central business districts of the smaller cities are bigger traffic generators per 1,000 population than are the larger cities. This, no doubt, is due largely to factors previously mentioned and to the fact that in the smaller cities the principal through routes pass through the downtown area with few or no alternates available.

Table 6.—*Traffic volumes and ratios in the central business district*

Population group	Number of cities	Average 8-hour volume, inbound [1]	Average hour volume, in and out	Peak half-hour volume, in and out [2]	Ratio of peak to average hour, in and out	In-and-out volume per 1,000 population, peak half-hour	Percentage of vehicles passing through [3] in—	
							8 hours	Peak half-hour
Under 25,000	9	13,000	3,300	2,500	1.34	139	52	59
25,000–50,000	4	21,000	5,300	3,600	1.35	107	61	70
50,000–100,000	2	26,000	6,600	4,420	1.34	61	52	69
100,000–250,000	7	41,000	11,000	6,810	1.33	41	60	70
250,000–500,000	3	54,000	13,700	9,110	1.33	25	60	75
Over 500,000	2	69,000	17,600	12,000	1.34	14	58	91

[1] 8-hour period, 10 a. m. to 6 p. m.; all vehicles.
[2] Peak half-hour for traffic movement, generally between 5 and 6 p. m.
[3] Percentage of vehicles entering the central business district.

Table 7.—*Length of time parked and distance walked to destination*

Population group	Number of cities	Percentage parked—		Percentage walking—	
		Less than 30 minutes	4 hours and over	Less than 400 feet	800 feet and over
Under 25,000	9	56	8	70	9
25,000–50,000	4	52	10	78	5
50,000–100,000	2	52	10	77	7
100,000–250,000	8	43	15	64	14
250,000–500,000	2	34	20	63	19
Over 500,000	2	[1] 28	25	46	30

[1] Estimated from different groupings of length of time parked.

Regardless of the size of the city, the ratio of peak-hour traffic to average hourly traffic in the 8-hour period is practically constant. Peak-hour volumes are about one-third again as large as the volumes during the average hour of the survey period.

The proportion "passing through" the central business district may more correctly be described as those who do not stop to park. It includes whatever "cruisers" there may be and, additionally, those cars in service stations or in garages being serviced or repaired. These data, in the last two columns of table 6, refer to vehicles entering the central business district and not to vehicles leaving or to number of trips. There does not seem to be much difference in the proportion of traffic passing through the central business district during the business day (10 a. m. to 6 p. m.) in cities of different population groups.

The proportion of traffic entering the central business districts in the peak half-hour of traffic movement (usually between 5 and 6 p. m.), which does not stop to park, increases as the size of the city increases. The development of employment centers in sections of the city other than the central business district creates a large movement of population twice a day going to and coming from work. Much of this movement is across town and through the district.

Figure 4 shows the volume of traffic entering and leaving the central business district, and the hourly accumulation of parked vehicles and of all vehicles in the central business district, in a typical city of 175,000 population. The bars for each street around the periphery of the cordon show the total 8-hour volume of traffic entering and leaving the central business district, segregated by type of vehicle and by direction as indicated in the legend for street traffic at the left.

The bar chart within the cordon line in figure 4 shows the hourly total inbound and outbound traffic volumes, the shaded lower portions of the bars indicating the proportion of inbound traffic that entered the central business district and parked, and the proportion of outbound traffic coming from parking spaces. Plotted across the bar chart are curves showing the accumulation of traffic and of parking.

The difference between the volume of inbound traffic and outbound traffic for each hour period, when cumulated, represents the number of vehicles remaining in the business district at any hour period. When allowance is made for the number of vehicles parked, the number moving on the streets at the beginning of the study period, and the number of parkers with origins within the district, it is possible to estimate the actual volume of inbound traffic which may be considered as potential users of an alternate route.

Continued periodic cordon counts are desirable, after the completion of a study, to detect changing trends in traffic movements and in the accumulation of vehicles in the downtown area, the latter being an indication of the demand for parking space. By repeating the cordon count, the value of the basic parking study can be extended over a longer period of time.

Length of Time Parked

Data have hitherto been available from many individual studies concerning the habits of parkers as to duration of parking and the distances walked after parking, but this is the first time it has been possible to assemble the results of studies made under the same procedures in cities of various population groups.

The proportion of cars parked less than 30 minutes in the largest cities is only half of those parked for the same length of time in the smallest cities, as shown in table 7. The proportion parked 4 hours and over, however, is three times as large. An influence here is the practice, more common in smaller cities, of using the car to drive home or elsewhere at noon for lunch.

Walking Distances

Definite trends are also apparent, from table 7, in the distances people walk to their destinations after parking their cars. The lengths of blocks vary but, generally speaking,

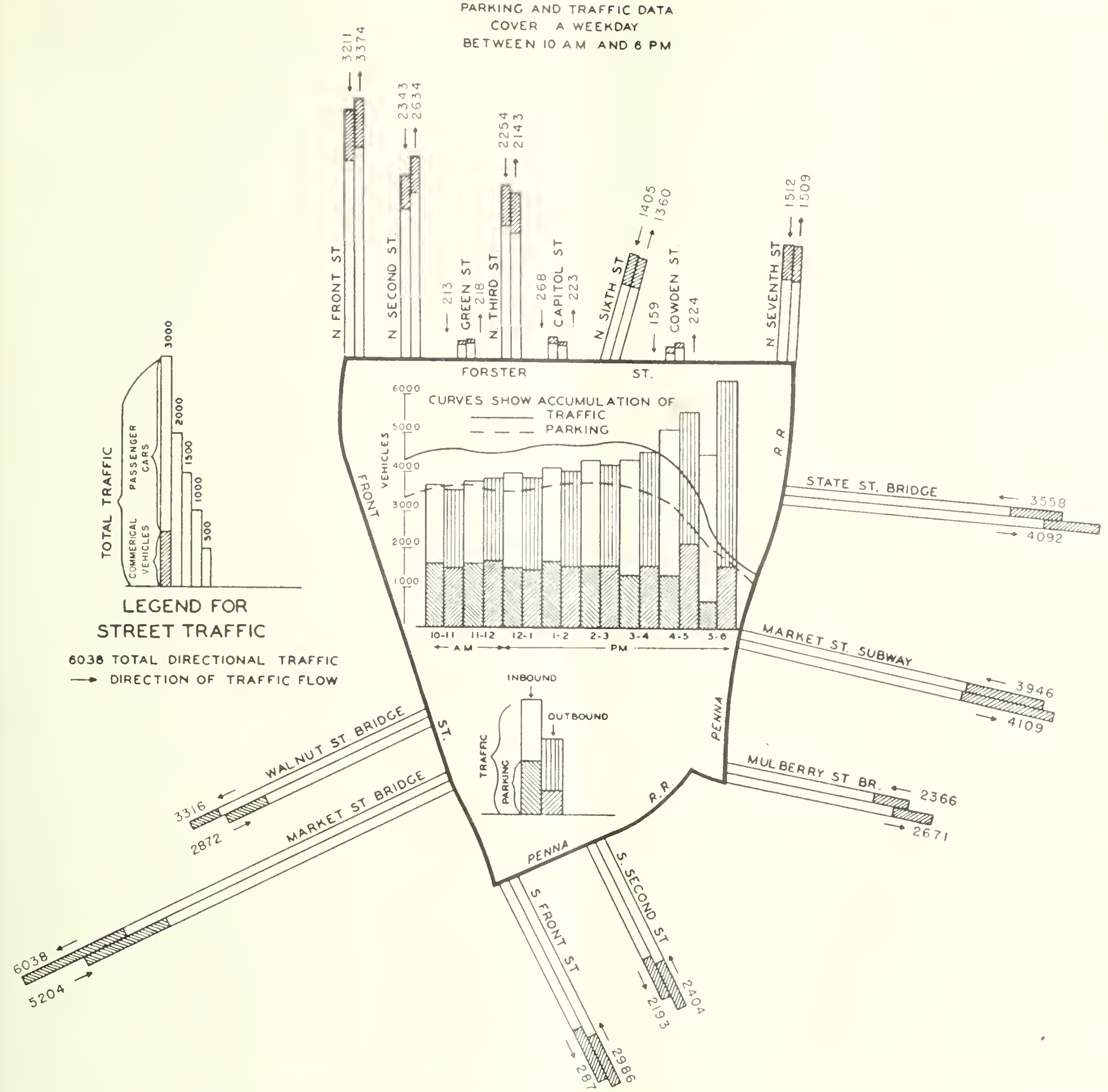

Figure 4.—Traffic entering or leaving the central business district and vehicles parking in that area, in a typical city of 175,000 population.

one block may be considered to be about 400 feet long. In small cities almost three-fourths of the people parking in the central business district park within one block of their destination. This proportion decreases to less than 50 percent in the largest cities.

The proportion walking more than 800 feet (two blocks) is relatively small in the smaller cities—less than 10 percent. In the largest cities, however, as many as 30 percent of the parkers walk more than 800 feet. These data may be useful, in a general way, in estimating the seriousness of the parking problem in other cities where extensive studies have not been made.

Trip Purposes

In any study in which an interview must be completed in less than a minute, it is extremely difficult to define trip purposes so definitely that they will be uniformly understood and applied. The results obtained, however, show definite trends for the average length of time parked for each of the more common trip purposes, as illustrated in table 8. Regardless of purpose, the average length of time parked increases in the larger cities as compared with the smaller cities. There does not appear to be much difference in the length of time parked by shoppers and by those on business trips. In both instances, the time parked increases with the size of the city.

"Other" purposes include trips for meals, and to doctors, dentists, movies, and social and other recreational activities. There does not seem to be much difference in the length of time parked for these purposes in cities of different sizes.

Effect of Parking Meters

The comprehensive type of parking study, wherein the type of parking and times of arrival and departure are noted, permits special types of analyses to be made—among these being observation of the extent of violation of parking-time restrictions shown in table 9.

All of the 25 cities for which pertinent data are available have one or more zones in the central business district with time restrictions but without parking meters, and 13 of them have metered zones. In table 9, the first group reported includes some cities in which there were no meters, some cities where there were no unmetered curbs, and some having both metered and unmetered curbs. It is, therefore, only a general comparison between the two types of zones.

To present data on a more nearly comparable basis, an analysis is shown in table 9 for a second group of cities, comprising only those where data were available in the same city and in the same time-restriction class for both metered and unmetered but restricted spaces.

Data from each of 11 cities, where curb parking was observed in both types of zones, indicate that violations in metered zones were considerably less, both as to the numbers of violators and as to their use of time. This is also true when the data are segregated in time-restriction groups. It is also apparent that, in general, the proportion of overtime parkers and of overtime usage decreases as the length of time restrictions increases.

It is probable that the data from these several cities are not strictly comparable because of differences in enforcement policies. On the other hand, all data were obtained from studies limited to the central business districts, where there was little difference in opportunity for enforcement. As the occasion presents itself, studies should be made before and after meters are installed to obtain as fair a comparison as possible of the violation of parking restrictions.

Table 8.—*Average length of time parked for each purpose of trip*

Population group	Number of cities	Average time parked for each trip purpose, in hours				
		Work	Shopping	Business	Other	All purposes
Under 25,000	9	3.0	0.7	0.7	1.0	1.1
25,000–50,000	4	2.9	.7	.8	.9	1.3
50,000–100,000	2	3.3	.8	.7	.9	1.3
100,000–250,000	8	4.4	1.1	1.2	1.7	1.8
250,000–500,000	3	4.5	1.4	1.2	1.5	1.8
Over 500,000	2	5.1	1.4	1.4	1.2	2.5

Table 9.—*Comparison of overtime parking at metered curbs and at unmetered but restricted curbs, in the central business district*

Time zones	Number of cities	Percentage of overtime parking [1]		Percentage of space hours—			
				Used by violators [2]		Used in excess [3]	
		Unmetered	Metered	Unmetered	Metered	Unmetered	Metered
ALL CITIES REPORTING							
All	25	28.4	--------	45.6	--------	31.3	--------
All	13	--------	21.9	--------	47.4	--------	27.3
CITIES WITH BOTH METERED AND UNMETERED SPACES							
All	11	37.6	24.3	58.0	50.4	40.2	28.1
15-minute	2	67.5	61.3	60.6	84.2	51.7	61.2
30-minute	2	57.6	33.3	87.2	62.0	71.2	39.1
60-minute	7	37.6	22.0	65.3	53.0	44.9	28.8
90-minute	1	33.8	18.6	65.0	53.6	42.9	30.4
2-hour	6	21.8	12.5	38.1	32.3	21.0	14.6

[1] Percentage of all curb parkers in zones indicated.
[2] Total usage, including legal.
[3] Overtime usage only.

STATE ROAD-USER AND PERSONAL-PROPERTY TAXES ON SELECTED MOTOR VEHICLES

(*Continued from page 39.*)

It might have been desirable to have included in this study a heavy passenger car and several additional trucks and combinations. The purpose, however, was to explore the field in a general way, without attempting the exhaustive investigation and analysis that would have been required to cover every possible vehicle type and every possible tax.

The relatively low gross vehicle weight of the 40,000-pound tractor-semitrailer was chosen in order to have a vehicle that could be registered in any State; and had it not been for this factor a combination of 50,000 to 56,000 pounds, in general use in many States, might have been included.

A diesel-powered vehicle of 70,000 to 75,000 pounds gross vehicle weight might also have been represented but the differences between gasoline and diesel vehicles in fuel consumption, fuel- and vehicle-tax schedules, etc., make comparisons difficult; and the very heavy diesel units, common throughout the West, exceed the maximum size and weight limits of most of the other States. Furthermore, the diesels now in use are engaged primarily in interstate operation and this study was limited to intrastate vehicles. The use of diesel power in heavy vehicles, however, has already become sufficiently important to warrant special study of existing road-user taxes with respect to their application to diesel vehicles.

THE IDENTIFICATION OF ROCK TYPES

(*Continued from page 47.*)

amphibolites contain plagioclase feldspar instead of quartz. This distinction may be impossible to make in the hand specimen.

Difficulties with Intrusive Igneous Rocks

In trials of this method by selected groups of students, more difficulty was found in identification of the intrusive type of igneous rocks than any other general group. The difficulty may have been due to too much emphasis on the feldspar content of the rock, and too little regarding the other essential constituents. A study of figure 1 will show that of the coarse-grained igneous rocks, granite is the only kind which contains an appreciable amount of quartz, and that gabbro contains the greatest amount of the dark ferromagnesian minerals (mica, hornblende, and augite) with diorite containing the next greatest amount. Consequently, if the rock is hard, with visible, interlocking grains of approximately the same size, and contains an appreciable amount of quartz, it probably is a granite. On the other hand, if the rock has the same characteristics as given above, but contains little if any quartz, it can be named by reference to the ferromagnesian mineral content as indicated by the color of the rock. A rock of light color may be a syenite, one of medium color a diorite, and one of dark color a gabbro.

Attention should be given to the transition of rock by insensible stages from one kind to another. Granites will grade into syenites, and syenites into diorites, for instance; or a coarse-grained rock will grade into a fine-grained rock of the same mineralogical composition. Two samples of rock from the same deposit may be sufficiently different to warrant different names. Consequently, the identification of the hand specimen by the method given here does have some limitations. However, a careful study of the sample, following the method outlined, should permit the user to name the rock with a relatively small margin of error.

State Road-User and Personal-Property Taxes on Selected Motor Vehicles, 1950

*BY THE RESEARCH REPORTS BRANCH
BUREAU OF PUBLIC ROADS*

**Reported by R. W. MEADOWS and
S. F. BIELAK, Highway Economists**

The extensive need for highway improvement with which the nation is faced has raised to vital importance the study of existing and prospective sources of highway revenue. Road-user imposts—registration fees and gasoline taxes—supply a major part of the current income for highways. But in considering the levies paid on motor vehicles, direct property taxes must also be taken into account. Although not collected as highway revenue, these property taxes are in many States assessed in sizable amount and in such different fashion from taxes on other forms of property as to make them particular levies on motor vehicles.

In this report there are presented data on the registration and other fees, gasoline taxes, and direct property taxes that would be collected for each of seven representative types of vehicles in each of the States during 1950. In addition, data are reported separately for some of the commercial vehicles according to their use in private operation, farm service, or contract hauling. Comparisons can thus be made of the taxes paid for similar vehicles in different States, for different types of vehicles in the same State, or for the same vehicle in different types of service.

The ranges discovered among the States are extensive. For instance, the registration fee for a three-axle combination in private use varies from $45 to $577, and the property tax from $35 to $235. The total taxes on a single-unit van in contract service range from $135 to $746. The average for all States of road-user plus property taxes for a light passenger car is $55; for a five-axle combination in contract service it is $2,400. For a single-unit stake truck, the average of total taxes is $69 for farm service, $132 for private use, and $217 for contract carriage.

THIS study presents a comparison of the State road-user taxes and all direct property taxes that would be paid on a group of typical motor vehicles in one year. It is intended primarily to show the road-user taxes on a specific group of vehicles and to provide comparisons between the taxes paid on (1) different vehicles in a given State, (2) similar vehicles in different States and regions, (3) similar vehicles in different types of service in the same State, and (4) similar vehicles in different types of service in different States and regions. The information presented makes it possible to study the principal road-user taxes as a group, in preference to considering each separately as has frequently been necessary in the past. All of the taxes and fees given are for a full year, based on tax rates in effect January 1, 1950.

The Typical Vehicles

Seven vehicles and combinations, representing a cross section of the vehicle population, were selected for inclusion in the study—one light-weight passenger car, one medium-weight passenger car, three single-unit trucks, and two tractor-semitrailer combinations. The single-unit trucks include a half-ton pickup, a 1½-ton stake truck, and a 2½-ton van. Of the two tractor-semitrailer units, one is a three-axle combination of 40,000 pounds gross vehicle weight, chosen so that it would fall within the maximum length and weight limits of all of the States. The other combination is a five-axle tractor-semitrailer of 64,000 pounds gross vehicle weight (indicative of heavy operation) which would be permitted to register in 24 States.

In order to obtain the tax data on a uniform basis in all of the States, a detailed outline of the specifications of each of the selected vehicles was submitted to the State authorities in November 1949, for determination of the exact fees and taxes. Table 1 gives these specifications, together with the assumed annual mileage traveled, motor fuel consumed, and other factors needed to compute the various taxes. It was also specified that the vehicles were in intrastate operation, and that they had been operated in the same State since purchased new in 1948. This avoided the complex situations that would have been encountered in computing taxes on vehicles in interstate operation, and eliminated consideration of the sales taxes (or their counterparts) on new vehicles that are in effect in most States. The relative sizes of the seven typical vehicles are represented on this page (only one passenger car is shown since the two vehicles in this category are very nearly the same length).

Types of Service

Distinctly different tax rates are in effect in most States for commercial vehicles operated privately (not for hire), as contract carriers (for hire), and in farm service. Information was therefore obtained, and was tabulated separately, for each of these three types of operation, where applicable, for each vehicle studied.

Vehicles in private operation are those used solely for carrying goods owned or sold by the vehicle owner, with no direct transportation charge. Contract carriers are employed in hauling goods for others, with a direct transportation charge, at times and to destinations according to the jobs for which they are hired. Operation of contract carriers is usually subject to State franchise and regulation. Common carriers, which operate for hire over established routes and on fixed schedules, are not included in the study because of their more complex tax schedules and forms of regulation, and the fact that their operation is frequently interstate.

The farm service classification represents a reduced registration fee for farmers' trucks, but without restriction as to the highways they may use. This arrangement is in effect in 23 States. (The nominal charge imposed as the sole registration fee by some States for farm vehicles licensed for restricted operation on or in the immediate vicinity of the farm is not included in this study.)

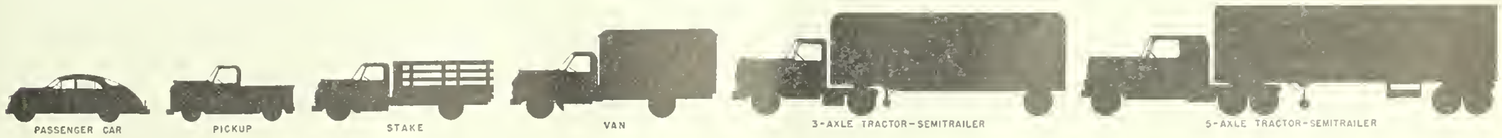

Table 1.—*Outline of data used in computing State road-user taxes and property taxes on selected vehicles, 1950 registration year*

Factors affecting taxation	Vehicle description and use (vehicle purchased new in 1948)								
	Passenger car		Single-unit truck			Three-axle tractor-semitrailer combination		Five-axle tractor-semitrailer combination	
	Light two-door sedan	Medium four-door sedan	Pick-up	Stake	Van	Tractor-truck	Semi-trailer, 28-foot van	Tractor-truck	Semi-trailer, 34-foot van
Price, f. o. b. factory	$1,307	$2,087	$1,164	$1,610	$4,450	$5,000	$3,265	$15,400	$5,900
Horsepower:									
S. A. E.	25.35	30.63	32.5	29.4	33.75	36.0	------	72.6	------
Brake	95.0	115.0	100.0	90.0	111.5	110.0	------	245.0	------
Weight:									
Manufacturer's rated capacity ... tons	------	------	½	1½	2½	2	------	------	------
Chassis weight ... pounds	------	------	------	4,180	5,700	------	------	------	------
Empty weight ... do	2,995	3,855	3,061	5,140	7,900	8,000	7,500	19,000	8,900
Load capacity ... do	------	------	1,640	7,360	10,600	14,000	10,500	17,000	19,100
Gross vehicle weight ... do	3,895	4,755	4,700	12,500	18,500	22,000	18,000	36,000	28,000
Gross weight of combination ... pounds	------	------	------	------	------	40,000	------	64,000	------
Length:									
Wheelbase ... inches	------	------	114	161	172	136	------	203	------
Distance between tandem axles ... inches	------	------	------	------	------	------	------	48	48½
Length of combination ... feet	------	------	------	------	------	39½	------	49½	------
Tires:									
Size	------	------	------	7.50×20	9.00×20	10.00×20	10.00×20	11.00×20	11.00×20
Arrangement	Single	Single	Single	Dual	Dual	Dual	Dual	Dual tandem	Dual tandem
Total annual mileage:									
Farm ... miles	------	------	6,000	5,500	------	------	------	------	------
Not for hire ... do	9,500	10,500	9,000	12,000	15,000	35,000	------	------	------
For hire: Contract carrier ... miles	------	------	------	20,000	25,000	45,000	------	75,000	------
Average mileage per gallon:									
Farm	------	------	15.5	10.0	------	------	------	------	------
Not for hire	16.5	14.5	15.0	9.0	6.5	5.0	------	------	------
For hire: Contract carrier	------	------	------	9.0	6.5	5.0	------	3.0	------
Annual motor-fuel consumption:									
Farm ... gallons	------	------	387	550	------	------	------	------	------
Not for hire ... do	576	724	600	1,333	2,307	7,000	------	------	------
For hire: Contract carrier ... gallons	------	------	------	2,222	3,846	9,000	------	25,000	------
Total revenue ton-miles (avg. load)	------	------	------	40,400	73,000	303,300	------	744,750	------
Average gross weight ... pounds	------	------	------	9,180	13,730	[1] 28,975	------	[1] 47,755	------
Gross annual earnings	------	------	------	$7,000	$10,000	$20,000	------	$40,000	------

[1] Gross weight of combination.

Property Taxes

In most cases the property taxes on motor vehicles have little or no relation to the use of highways and the revenues from them are not available for highways. They are so closely associated with registration fees in their relation to costs of operation, however, and comprise such a large portion of the total taxes paid on motor vehicles in some States that their inclusion in this study was necessary in order to obtain equitable comparisons. The registration fees for individual vehicles vary considerably among the States, and the property taxes in some instances greatly exceed the registration fees paid. When property taxes and registration fees are considered together, however, the differences among the States are proportionately smaller. It is worth noting that the valuation of motor vehicles for tax purposes is generally conceded to be much closer to market value than is the valuation of other personal property subject to the same taxes. It is also probable that motor vehicles constitute a very considerable portion of taxable personal property in many jurisdictions. While this study is not directly concerned with personal property taxes per se, it suggests an area of research that should not be neglected by those interested in the tax burdens on the motor-vehicle owner.

The property taxes that are given, for all except the farm vehicles, include all taxes levied by the State, county, city, or other local governmental unit in which the vehicle is domiciled, and are the taxes that would be imposed on the vehicle in the capital city of the State. The property taxes given for farm vehicles include State, county, and district or other property taxes that would be collected in an average rural agricultural community in the State. In a few States where uniform State-wide valuation and tax rates are in effect in all jurisdictions, there would be no difference between property taxes on farm vehicles and those registered in the capital city of the State. In two States—California and Washington—property taxes on motor vehicles have been replaced by "in lieu" taxes. These taxes have many of the characteristics of property taxes, but since neither the levies nor the distribution of their proceeds are directly related to the jurisdiction in which they were collected, they have been classified as road-user imposts. In both of these instances the State courts have ruled them not to be property taxes.

Carrier Taxes

In the consideration of motor-carrier taxes paid for vehicles in contract-carrier service, gross receipts taxes and other levies that are imposed on general business, and not limited to motor carriers, have been eliminated. Fees for authority to operate, and other fees paid only at the time a carrier begins to haul goods for hire, have also been excluded.

Care should be taken in comparing the carrier taxes of the several States. Many of the States have special registration classes for vehicles used in for-hire service. The fees in the special registration classes, in most cases, are substantially greater than the registration fees for vehicles in private (not-

Table 2.—*Average, low, and high road-user and property taxes on selected motor vehicles*

Vehicle and service	Average fee for all States					Lowest fee					Highest fee				
	Registration fee [1]	Gasoline tax	Total road-user taxes	Property tax	Total	Registration fee [1]	Gasoline tax	Total road-user taxes	Property tax	Total	Registration fee [1]	Gasoline tax	Total road-user taxes	Property tax	Total
Passenger car:															
Light-weight	$11.53	$29.33	$40.85	$24.07	$55.10	$2.50	$11.52	$20.28	$5.00	$27.78	$24.00	$51.84	$61.84	$41.62	$85.90
Medium-weight	14.90	36.86	51.77	34.36	72.11	3.00	14.48	25.48	8.00	35.37	39.00	65.16	81.18	62.76	114.20
Pick-up truck:															
Farm	13.09	19.71	32.80	14.76	41.23	2.50	7.74	17.74	5.00	20.48	37.00	34.83	54.42	35.10	64.69
Private	16.48	30.55	47.03	21.10	59.52	2.50	12.00	25.50	5.00	39.00	37.00	54.00	69.70	35.02	89.45
Stake truck:															
Farm	29.22	28.01	57.22	19.78	68.52	4.50	11.00	21.00	5.00	32.44	70.00	49.50	99.00	37.75	125.08
Private	48.07	67.89	115.95	27.55	132.26	10.00	26.67	56.67	14.49	73.11	132.50	120.00	185.83	48.42	208.42
Contract	87.46	113.15	200.61	27.55	216.91	15.00	44.44	74.44	14.49	90.88	342.50	200.00	498.05	48.42	498.05
Van truck:															
Private	93.62	117.47	211.09	68.32	251.53	25.00	46.14	86.14	18.50	104.64	272.50	207.63	364.78	233.50	505.41
Contract	152.32	195.83	348.15	68.32	388.59	32.00	76.92	116.92	18.50	135.42	476.50	346.14	745.72	233.50	745.72
Three-axle combination:															
Private	229.28	356.43	585.70	121.65	657.70	45.00	140.00	271.00	34.94	327.94	577.00	630.00	997.00	234.56	1,043.25
Contract	373.50	458.27	831.76	121.65	903.76	66.00	180.00	333.00	34.94	367.94	892.00	810.00	1,441.00	234.56	1,441.00
Five-axle combination:															
Contract	920.65	1,322.91	2,243.56	289.53	2,400.40	217.00	750.00	1,217.00	59.38	1,398.75	2,186.50	2,250.00	3,936.50	521.34	3,936.50

[1] Includes the carrier taxes and "other taxes and fees" shown in tables 3–8.

Table 3.—*Road-user and personal-property taxes on selected passenger vehicles*

Region and State	Light two-door sedan							Medium four-door sedan						
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Rank of State		Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Rank of State	
						Total fees and taxes	Total, excluding property tax						Total fees and taxes	Total, excluding property tax
New England:														
Maine	$14.00	$16.34	-----	$34.56	$64.90	14	10	$16.00	$26.09	-----	$43.44	$85.53	13	14
New Hampshire	13.65	11.76	-----	23.04	48.45	29	34	21.60	18.78	-----	28.96	69.34	27	26
Vermont	22.00	------	-----	28.80	50.80	28	7	22.00	------	-----	36.20	58.20	37	16
Massachusetts	3.00	23.09	-----	17.28	43.37	38	49	4.50	36.86	-----	21.72	63.08	30	48
Rhode Island	12.00	26.60	-----	23.04	61.64	17	35	16.00	33.60	-----	28.96	78.56	17	36
Connecticut	7.00	36.37	-----	23.04	66.41	11	41	9.00	47.99	-----	28.96	85.95	12	41
Middle Atlantic:														
New York	15.00	------	-----	23.04	38.04	44	32	20.50	------	-----	28.96	49.46	42	28
New Jersey	10.00	------	$1.00	17.28	28.28	48	44	15.50	------	$1.00	21.72	38.22	48	40
Pennsylvania	10.00	------	-----	28.80	38.80	43	30	10.00	------	-----	36.20	46.20	45	34
Delaware	8.00	------	-----	28.80	36.80	46	33	12.00	------	-----	36.20	48.20	44	32
Maryland	10.00	5.00	-----	28.80	43.80	36	29	15.00	8.00	-----	36.20	59.20	35	25
District of Columbia	5.00	18.00	1.00	23.04	47.04	32	42	8.00	25.00	1.00	28.96	62.96	31	42
West Virginia	17.00	32.65	-----	28.80	78.45	5	16	22.40	52.15	-----	36.20	110.75	2	15
Southeastern:														
Virginia	9.00	22.00	-----	34.56	65.56	13	21	11.70	28.60	-----	43.44	83.74	14	21
North Carolina	10.00	19.79	-----	40.32	70.11	10	9	12.00	32.68	-----	50.68	95.36	7	9
South Carolina	3.00	25.80	1.00	34.56	64.36	15	31	4.00	39.12	1.00	43.44	87.56	11	30
Georgia	2.50	28.48	-----	40.32	71.30	9	22	4.50	39.61	-----	50.68	94.79	8	20
Florida	15.00	------	.25	40.32	55.57	24	4	20.00	------	.25	60.68	70.93	26	5
Kentucky	4.50	36.19	.50	40.32	81.51	3	18	4.50	54.75	.50	60.68	110.43	3	19
Tennessee	7.50	------	.50	40.32	48.32	30	11	10.00	------	.50	50.68	61.18	33	12
Alabama	13.00	18.09	.50	34.56	66.15	12	12	18.00	21.44	.50	43.44	83.38	15	11
Mississippi	10.22	41.12	-----	34.56	85.90	1	19	12.70	51.20	-----	43.44	107.34	5	18
East Central:														
Ohio	10.00	------	-----	23.04	33.04	47	38	10.00	------	-----	28.96	38.96	47	39
Indiana	11.00	27.18	-----	23.04	61.22	18	37	12.00	36.17	-----	28.96	77.13	20	37
Illinois	10.50	15.01	-----	17.28	42.79	39	47	10.50	19.05	-----	21.72	51.27	40	47
Michigan	10.50	------	-----	17.28	27.78	49	46	13.65	------	-----	21.72	35.37	49	44
Wisconsin	16.00	------	-----	23.04	39.04	42	27	16.00	------	-----	28.96	44.96	46	35
Minnesota	23.20	------	-----	28.80	52.00	26	6	39.00	------	-----	36.20	75.20	22	3
Iowa	24.00	------	-----	23.04	47.04	33	15	33.00	------	-----	28.96	61.96	32	10
Missouri	11.00	15.41	-----	11.52	37.93	45	48	11.00	24.66	-----	14.48	50.14	41	49
Southwestern:														
Arkansas	13.00	21.88	-----	37.44	72.32	8	8	18.00	28.30	-----	47.06	93.36	10	7
Louisiana	3.00	22.62	-----	51.84	77.46	6	5	3.00	28.28	-----	65.16	96.44	6	6
Oklahoma	23.90	------	.50	37.44	61.84	16	1	33.62	------	.50	47.06	81.18	16	1
Texas	11.16	21.36	-----	23.04	55.56	25	36	19.20	26.82	-----	28.96	74.98	24	33
West Central:														
North Dakota	22.50	------	-----	23.04	45.54	34	17	30.75	------	-----	28.96	59.71	34	13
South Dakota	17.00	------	-----	23.04	40.04	40	25	25.00	------	-----	28.96	53.96	38	22
Nebraska	8.00	39.25	-----	34.56	81.81	2	23	8.00	62.76	-----	43.44	114.20	1	24
Kansas	10.00	41.62	-----	28.80	80.42	4	28	13.15	59.12	-----	36.20	108.47	4	29
Mountain:														
Montana	13.00	25.80	-----	34.56	73.36	7	14	13.00	37.04	-----	43.44	93.48	9	17
Idaho	5.00	------	-----	34.56	39.56	41	26	5.00	------	-----	43.44	48.44	43	31
Wyoming	5.00	15.68	-----	23.04	43.72	37	45	5.00	25.04	-----	28.96	59.00	36	46
Colorado	5.60	15.68	-----	34.56	55.84	23	24	6.95	25.04	-----	43.44	75.43	21	27
New Mexico	18.00	------	-----	40.32	58.32	20	2	27.00	------	-----	50.68	77.68	18	2
Arizona	3.50	18.60	-----	28.80	50.90	27	39	3.50	28.20	-----	36.20	67.90	28	38
Utah	5.00	30.12	.50	23.04	58.66	19	43	5.00	42.72	.50	28.96	77.18	19	45
Nevada	5.00	26.50	-----	25.92	57.42	22	40	5.00	37.50	-----	32.58	75.08	23	43
Pacific:														
Washington	5.00	------	15.75	37.44	58.19	21	3	5.00	------	22.25	47.06	74.31	25	4
Oregon	10.00	------	-----	34.56	44.56	35	20	10.00	------	-----	43.44	53.44	39	23
California	6.00	------	16.00	25.92	47.92	31	13	6.00	------	25.00	32.58	63.58	29	8

for-hire) service. The difference, while technically classed as part of the registration fee, is in reality a special carrier fee. There is, of course, the compensating factor that vehicles in carrier service usually can be expected to operate a considerably greater mileage than those not for hire, thus reducing the per-mile cost of their registration fees.

The majority of trucks are used for the ordinary private service of carrying goods owned or sold by the vehicle owner, and for which there is no direct transportation charge. Even these trucks, however, are subject to mileage or other carrier taxes under some conditions, and the variations in the tax structures of the States made it necessary in a few instances to make arbitrary decisions on the inclusion or exclusion of these special levies. The general policy followed was to include only the taxes that would have to be paid on almost all of the vehicles in a given group. For example, the laws of some States are so written that practically all vehicles of more than 1½ tons (manufacturer's rated capacity) would have to pay certain mileage or compensatory taxes, whether in private or for-hire operation. In the same States many vehicles of 1½-ton capacity or less would not be subject to the tax. In such a case the tax has been shown only for the heavier vehicles. It should be remembered that the purpose of this study is to present a comparison of tax rates and burdens on certain typical vehicles rather than to include all taxes on all vehicles. Undoubtedly many of the larger trucks traveling in interstate commerce are subject to greater taxes than are reported in this study, and are also subject to restrictions and regulations that impose additional burdens. The larger units apparently have certain offsetting economic advantages, however, or they would not be in use.

Data Collected From States

The data received from the States are presented in tables 3–8. Table 3 reports the taxes on the light- and medium-weight passenger cars. Table 4 shows the taxes on pick-up trucks in farm service and in private use: these light trucks are almost never registered for contract-carrier service. Table 5 covers the taxes on the single-unit stake truck, operated as a farm vehicle and in both private and for-hire hauling. Tables 6 and 7 report respectively the taxes on the single-unit van and the three-axle, 40,000-pound tractor-semitrailer in private and in contract-carrier service. These two types are seldom used on farms. Table 8 shows the taxes on the five-axle, 64,000-pound tractor-semitrailer operated as a contract carrier. Such large vehicles are rarely found in private or farm service.

In addition to the tabular presentation, the variations among States of the registration fees, gasoline taxes, and personal-property taxes, and their totals, are shown graphically in figures 1–6 (pp. 41–3) for some of the vehicles. Figures 1 and 2 represent average combined data for the two passenger cars; figures 3 and 4, for the single-unit, 1½-ton stake truck in private use; and figures 5 and 6, for the three-axle, 40,000-pound tractor-semitrailer in private use. In all six of these charts the States are arranged in order from lowest to highest costs, the rank in figures 1, 3, and 5 being according to total amount while in figures 2, 4, and 6 the rank is according to road-user taxes only.

Variations in Taxes

Figure 1 shows that for passenger cars the total road-user and personal-property taxes

Table 4.—*Road-user and personal-property taxes on a single-unit pick-up truck, 4,700 pounds gross vehicle weight*

Region and State	Farm							Private						
						Rank of State							Rank of State	
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax
New England:														
Maine	$15.00	$14.55	-----	$23.22	$52.77	5	11	$15.00	$14.55	-----	$36.00	$65.55	14	16
New Hampshire	21.15	10.48	-----	15.48	47.11	15	15	21.15	10.48	-----	24.00	55.63	31	26
Vermont	22.00	-----	-----	19.35	41.35	25	5	22.00	-----	-----	30.00	52.00	34	13
Massachusetts	7.50	20.56	-----	11.61	39.67	28	48	7.50	20.56	-----	18.00	46.06	40	49
Rhode Island	24.80	15.14	-----	15.48	55.42	3	7	18.00	21.20	-----	24.00	63.20	19	37
Connecticut	11.70	18.16	-----	15.48	45.34	19	38	11.70	25.05	-----	24.00	60.75	21	43
Middle Atlantic:														
New York	24.80	-----	-----	15.48	40.28	27	8	24.80	-----	-----	24.00	48.80	37	20
New Jersey	12.00	-----	$1.00	11.61	24.61	47	41	24.00	-----	$1.00	18.00	43.00	45	33
Pennsylvania	16.50	-----	-----	19.35	35.85	35	18	16.50	-----	-----	30.00	46.50	39	22
Delaware	10.00	-----	-----	19.35	29.35	44	34	15.00	-----	-----	30.00	45.00	42	29
Maryland	12.00	5.00	-----	19.35	36.35	34	27	12.00	5.00	-----	30.00	47.00	38	38
District of Columbia	20.00	14.66	1.00	15.48	51.14	6	16	20.00	14.66	1.00	24.00	59.66	22	27
West Virginia	15.00	23.28	-----	19.35	57.63	2	22	15.00	29.10	-----	30.00	74.10	7	28
Southeastern:														
Virginia	12.00	15.40	-----	23.22	50.62	7	20	12.00	15.40	-----	36.00	63.40	18	21
North Carolina	10.00	7.19	-----	27.09	44.28	21	14	20.00	16.26	-----	42.00	78.26	6	6
South Carolina	5.00	15.51	1.00	23.22	44.73	20	35	5.00	28.38	1.00	36.00	70.38	10	36
Georgia	2.50	35.10	-----	27.09	64.69	1	33	2.50	34.71	-----	42.00	79.21	5	30
Florida	23.25	-----	.25	27.09	50.59	8	2	23.25	-----	.25	42.00	65.50	15	2
Kentucky	4.50	16.00	.50	27.09	48.09	11	26	10.00	30.00	.50	42.00	82.50	3	12
Tennessee	7.50	-----	.50	27.09	35.09	37	21	15.00	-----	.50	42.00	57.50	29	10
Alabama	15.00	11.34	.75	23.22	50.31	10	10	15.00	18.09	.75	36.00	69.84	11	14
Mississippi	7.00	13.99	-----	23.22	44.21	22	32	10.00	34.69	-----	36.00	80.69	4	24
East Central:														
Ohio	15.70	-----	-----	15.48	31.18	42	29	26.50	-----	-----	24.00	50.50	35	18
Indiana	15.00	15.54	-----	15.48	46.02	16	31	15.00	19.15	-----	24.00	58.15	26	41
Illinois	12.00	9.77	-----	11.61	33.38	40	42	12.00	13.24	-----	18.00	43.24	44	46
Michigan	10.85	-----	-----	11.61	22.46	48	44	24.80	-----	-----	18.00	42.80	47	35
Wisconsin	5.00	-----	-----	15.48	20.48	49	47	20.00	-----	-----	24.00	44.00	43	32
Minnesota	20.00	-----	-----	19.35	39.35	29	9	25.00	-----	-----	30.00	55.00	33	11
Iowa	25.00	-----	-----	15.48	40.48	26	6	25.00	-----	-----	24.00	49.00	36	19
Missouri	10.00	10.72	-----	7.74	28.46	45	49	15.00	15.41	-----	12.00	42.41	48	48
Southwestern:														
Arkansas	12.00	10.34	-----	25.16	47.50	13	13	12.00	14.68	-----	39.00	65.68	13	17
Louisiana	3.00	-----	-----	34.83	37.83	31	12	10.00	25.45	-----	54.00	89.45	1	4
Oklahoma	9.60	-----	.50	25.16	35.26	36	19	20.00	-----	.50	39.00	59.50	23	7
Texas	9.40	7.06	-----	15.48	31.94	41	40	18.80	16.69	-----	24.00	59.49	24	34
West Central:														
North Dakota	21.00	-----	-----	15.48	36.48	33	17	34.00	-----	-----	24.00	58.00	27	8
South Dakota	15.00	-----	-----	15.48	30.48	43	30	15.00	-----	-----	24.00	39.00	49	40
Nebraska	8.00	16.07	-----	23.22	47.29	14	28	15.00	35.02	-----	36.00	86.02	2	15
Kansas	7.50	20.95	-----	19.35	47.80	12	39	7.50	33.03	-----	30.00	70.53	9	42
Mountain:														
Montana	10.00	12.43	-----	23.22	45.65	18	25	10.00	25.80	-----	36.00	71.80	8	23
Idaho	5.00	-----	-----	23.22	28.22	46	37	10.00	-----	-----	36.00	46.00	41	25
Wyoming	5.00	13.96	-----	15.48	34.44	38	46	5.00	13.96	-----	24.00	42.96	46	47
Colorado	5.75	13.97	-----	23.22	42.94	24	36	5.75	13.97	-----	36.00	55.72	30	39
New Mexico	16.00	-----	-----	27.09	43.09	23	4	16.00	-----	-----	42.00	58.00	28	9
Arizona	14.35	16.80	-----	19.35	50.50	9	24	14.35	16.80	-----	30.00	61.15	20	31
Utah	7.50	14.73	.50	15.48	38.21	30	43	7.50	23.55	.50	24.00	55.55	32	45
Nevada	5.00	14.47	-----	17.42	36.89	32	45	5.00	27.00	-----	27.00	59.00	25	44
Pacific:														
Washington	10.50	-----	10.25	25.16	45.91	17	3	16.00	-----	10.25	39.00	65.25	16	3
Oregon	10.85	-----	-----	23.22	34.07	39	23	21.70	-----	12.00	36.00	69.70	12	1
California	16.00	-----	21.00	17.42	54.42	4	1	16.00	-----	21.00	27.00	64.00	17	5

vary widely. Peculiarly enough, the three States with the highest charges have registration fees that differ very little, and gasoline taxes that are only about 50 percent larger, than the three States with the lowest totals. Property taxes alone in the three highest States exceed the sum of the registration fees and gasoline taxes in the three lowest States, which have no property taxes. It will be noted generally that the States with very low registration fees have compensatingly large property taxes, while the States with high registration fees have no property taxes at all. A similar but less striking situation exists for trucks and combinations, as evidenced in figures 3 and 5.

On the other hand, it may be seen from figure 2 that, of the five States with gasoline-tax rates of 3 cents or less, Massachusetts, Missouri, and Illinois have the lowest road-user revenue per passenger car of all the States, while Michigan and New Jersey fare little better. The latter two States, collecting no personal property taxes, are the lowest in figure 1. The effect of the low gasoline-tax rates is similarly evident for trucks and combinations in figures 3–6.

Extremes and Averages

The road-user and personal-property taxes listed in detail for all States in tables 3–8 are summarized in table 2 (page 34) to show the nation-wide average fees for each of the vehicles selected, together with the lowest and highest fees for each vehicle and type of service. The average fees are simple unweighted arithmetic averages of the totals for all States. In the case of property taxes they are averages reflecting only the States that levy such a tax.

In making comparisons between national averages and the fees of a given State, or between fees for different vehicle types within the same or other States, adequate allowance should be made for the fact that some of the States with relatively high taxes on specific vehicles, or on vehicles in certain types of service, have a very small number of vehicles registered in that group.

Truck Registration Fees

The variations in truck registration fees among States, as shown in tables 4–8, are of considerable interest. In some States the tax schedules range from relatively low fees on light trucks to relatively high fees on heavy trucks (when compared with the fee ranges in other States), while in others this is reversed. The Mississippi registration fee for the pick-up truck, for example, ranked 38th among the States, but the fees for the heavier trucks in private service were 28th, 20th, and 9th, respectively, reflecting relatively more favorable fees for light trucks and less favorable fees for heavy trucks, when compared with other States. New Jersey, on the other hand, collects the 7th highest fee on the pick-up truck, with the fees on the heavier vehicles ranking 13th, 18th, and 29th among the States, thus imposing relatively high fees on light trucks as compared with heavy trucks. (In both of these States, the heavy tractor-semitrailer is not included because it is illegal.)

Table 5.—*Road-user and personal-property taxes on a single-unit stake truck, 12,500 pounds gross vehicle weight*

Region and State	Farm							Private							Contract, for hire							
						Rank of State							Rank of State								Rank of State	
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax
New England:																						
Maine	$60.00	$20.13	----	$33.00	$113.13	2	2	$60.00	$20.13	----	$80.00	$160.13	11	11	$60.00	$20.13	----	$5.00	$133.33	$218.46	21	24
New Hampshire	25.00	14.49	----	22.00	61.49	32	39	75.00	14.49	----	53.33	142.82	16	16	75.00	14.49	----	2.00	88.88	180.37	31	33
Vermont	25.00	------	----	27.50	52.50	41	31	106.25	------	----	66.67	172.92	7	4	106.25	------	----	------	111.11	217.36	22	18
Massachusetts	18.00	28.44	----	16.50	62.94	30	47	18.00	28.44	----	40.00	86.44	48	48	18.00	28.44	----	5.00	66.67	118.11	48	48
Rhode Island	39.00	20.94	----	22.00	81.94	12	14	39.00	29.32	----	53.33	121.65	26	36	39.00	29.32	----	5.00	88.88	162.20	38	41
Connecticut	37.50	25.11	----	22.00	84.61	7	17	37.50	31.71	----	53.33	122.54	25	41	37.50	31.71	----	5.00	88.88	163.09	37	42
Middle Atlantic:																						
New York	40.80	------	----	22.00	62.80	31	12	40.80	------	----	53.33	94.13	44	35	40.80	------	----	------	88.88	129.68	46	43
New Jersey	25.50	------	$1.00	16.50	43.00	46	43	51.00	------	$1.00	40.00	92.00	45	37	51.00	------	$1.00	------	66.67	118.67	47	45
Pennsylvania	45.00	------	----	27.50	72.50	19	7	45.00	------	----	66.67	111.67	38	25	45.00	------	----	10.00	111.11	166.11	35	32
Delaware	22.50	------	----	27.50	50.00	44	35	45.00	------	----	66.67	111.67	39	26	45.00	------	----	------	111.11	156.11	40	37
Maryland	10.00	5.00	----	27.50	42.50	47	45	35.00	15.00	----	66.67	116.67	33	30	35.00	15.00	----	------	111.11	161.11	39	39
Dist. of Columbia	35.00	19.82	1.00	22.00	77.82	16	20	35.00	19.82	1.00	53.33	109.15	41	43	35.00	19.82	1.00	------	88.88	144.70	44	44
West Virginia	25.00	32.20	----	27.50	84.70	6	29	25.00	40.25	----	66.67	131.92	21	38	50.00	40.25	----	13.50	111.11	214.86	23	29
Southeastern:																						
Virginia	19.50	22.00	----	33.00	74.50	17	30	19.50	22.00	----	80.00	121.50	27	32	19.50	22.00	----	70.00	133.33	244.83	14	14
North Carolina	30.00	9.99	----	38.50	78.49	15	10	60.00	22.60	----	93.31	175.91	6	8	120.00	22.60	----	------	155.55	298.15	8	6
South Carolina	65.00	26.08	1.00	33.00	125.08	1	1	65.00	47.72	1.00	80.00	193.72	3	9	65.00	47.72	1.00	60.00	133.33	307.05	5	8
Georgia	10.00	36.00	----	38.50	84.50	8	37	10.00	35.60	----	93.31	138.91	17	28	25.00	35.60	----	25.00	155.55	241.15	15	20
Florida	51.00	------	.25	38.50	89.75	4	4	51.00	------	.25	93.31	144.56	15	10	141.75	------	.25	200.50	155.55	498.05	1	1
Kentucky	4.50	21.10	.50	38.50	64.60	28	42	32.00	39.56	.50	93.31	165.37	8	17	32.00	39.56	.50	77.00	155.55	304.61	7	7
Tennessee	12.50	------	.50	38.50	51.50	42	32	25.00	------	.50	93.31	118.81	29	20	50.00	------	.50	12.50	155.55	218.55	20	16
Alabama	22.50	13.44	.75	33.00	69.69	23	24	22.50	21.44	.75	80.00	124.69	23	29	22.50	21.44	.75	101.00	133.33	279.02	9	9
Mississippi	21.40	9.33	----	33.00	63.73	29	26	37.00	34.13	----	80.00	151.13	13	21	73.00	34.13	----	7.00	133.33	247.46	13	19
East Central:																						
Ohio	32.80	------	----	22.00	54.80	39	25	61.00	------	----	53.33	114.33	36	24	61.00	------	----	20.00	88.88	169.88	33	30
Indiana	35.00	15.54	----	22.00	72.54	18	21	35.00	19.15	----	53.33	107.48	42	44	35.00	19.15	----	12.00	88.88	155.03	41	40
Illinois	50.00	16.60	----	16.50	83.10	10	11	50.00	22.52	----	40.00	112.52	37	42	50.00	22.52	----	------	66.67	139.19	45	46
Michigan	17.85	------	----	16.50	34.35	48	48	51.00	------	----	40.00	91.00	46	40	51.00	------	----	30.00	66.67	147.67	42	38
Wisconsin	27.50	------	----	22.00	49.50	45	36	110.00	------	----	53.33	163.33	9	5	110.00	------	----	106.00	88.88	304.88	6	5
Minnesota	31.20	------	----	27.50	58.70	35	18	50.00	------	----	66.67	116.67	32	22	50.00	------	----	7.50	111.11	168.61	34	31
Iowa	70.00	------	----	22.00	92.00	3	3	70.00	------	----	53.33	123.33	24	18	70.00	------	----	5.00	88.88	163.88	36	34
Missouri	10.00	11.44	----	11.00	32.44	49	49	30.00	16.44	----	26.67	73.11	49	49	30.00	16.44	----	------	44.44	90.88	49	49
Southeastern:																						
Arkansas	36.00	13.98	----	35.75	85.73	5	8	42.00	19.83	----	86.67	148.50	14	15	42.00	19.83	----	------	144.44	206.27	24	26
Louisiana	10.00	------	----	49.50	59.50	34	16	60.00	25.45	----	120.00	205.45	2	3	120.00	25.45	----	10.00	200.00	355.45	3	3
Oklahoma	22.40	------	.50	35.75	58.65	36	19	95.00	------	.50	86.67	182.17	5	2	95.00	------	.50	------	144.44	239.94	17	11
Texas	40.63	7.06	----	22.00	69.69	22	13	81.25	16.96	----	53.33	151.54	12	13	81.25	16.96	----	11.00	88.88	198.09	28	27
West Central:																						
North Dakota	28.50	------	----	22.00	50.50	43	33	50.50	------	----	53.33	103.83	43	27	50.50	------	----	50.00	88.88	189.38	29	25
South Dakota	37.50	------	----	22.00	59.50	33	15	37.50	------	95.00	53.33	185.83	4	1	37.50	------	----	95.00	88.88	221.38	19	15
Nebraska	12.00	22.22	----	33.00	67.22	26	40	80.00	48.42	----	80.00	208.42	1	7	80.00	48.42	----	10.00	133.33	271.75	10	13
Kansas	15.00	28.98	----	27.50	71.48	20	44	15.00	45.69	10.00	66.67	137.36	18	39	15.00	45.69	----	30.00	111.11	201.80	27	36
Mountain:																						
Montana	20.00	14.92	----	33.00	67.92	25	27	20.00	31.10	----	80.00	131.10	22	31	20.00	31.10	----	45.00	133.33	229.43	18	23
Idaho	20.00	------	----	33.00	53.00	40	28	35.00	------	----	80.00	115.00	35	23	45.00	------	----	------	133.33	178.33	32	28
Wyoming	15.00	19.32	----	22.00	56.32	38	46	15.00	19.32	----	53.33	87.65	47	47	15.00	19.32	----	58.90	88.88	182.10	30	35
Colorado	17.50	19.32	----	33.00	69.82	21	34	17.50	19.32	----	80.00	116.82	31	33	17.50	19.32	----	80.80	133.33	250.95	12	12
New Mexico	43.50	------	----	38.50	82.00	11	6	43.50	------	----	93.31	136.81	19	12	21.75	------	----	25.00	155.55	202.30	26	22
Arizona	29.00	23.00	----	27.50	79.50	14	22	29.00	23.00	----	66.67	118.67	30	34	29.00	23.00	----	175.00	111.11	338.11	4	4
Utah	25.00	19.64	.50	22.00	67.14	27	38	25.00	31.77	.50	53.33	110.60	40	46	25.00	31.77	.50	------	88.88	146.15	43	47
Nevada	18.90	37.75	----	24.75	81.40	13	41	18.90	37.75	----	60.00	116.65	34	45	18.90	37.75	----	99.05	100.00	255.70	11	17
Pacific:																						
Washington	17.50	------	15.75	35.75	69.00	24	9	30.00	------	15.75	86.67	132.42	20	14	30.00	------	15.75	15.00	144.44	205.19	25	21
Oregon	23.40	------	----	33.00	56.40	37	23	46.80	------	36.00	80.00	162.80	10	6	46.80	------	----	60.00	133.33	240.13	16	10
California	36.00	------	23.00	24.75	83.75	9	5	36.00	------	23.00	60.00	119.00	28	19	36.00	------	23.00	227.50	100.00	386.50	2	2

Table 6.—*Road-user and personal-property taxes on a single-unit van truck, 18,500 pounds gross vehicle weight*

Region and State	Private							Contract, for hire							
						Rank of State								Rank of State	
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax
New England:															
Maine	$125.00	$55.63	------	$138.42	$319.05	9	11	$125.00	$55.63	-----	$5.00	$230.76	$416.39	22	25
New Hampshire	111.00	40.05	------	92.28	243.33	23	25	111.00	40.05	-----	2.00	153.84	306.89	31	37
Vermont	175.75	------	------	115.35	291.10	14	6	175.75	------	-----	------	192.30	368.05	25	21
Massachusetts	27.00	78.60	------	69.21	174.81	44	48	27.00	78.60	-----	5.00	115.38	225.98	46	48
Rhode Island	54.00	81.00	------	92.28	227.28	29	44	54.00	81.00	-----	5.00	153.84	293.84	33	45
Connecticut	55.50	90.87	------	92.28	238.65	26	43	55.50	90.87	-----	5.00	153.84	305.21	32	44
Middle Atlantic:															
New York	63.20	------	------	92.28	155.48	47	40	63.20	------	-----	------	153.84	217.04	47	41
New Jersey	80.00	------	$1.00	69.21	150.21	48	42	80.00	------	$1.00	------	115.38	196.38	48	47
Pennsylvania	70.00	------	------	115.35	185.35	42	32	70.00	------	-----	10.00	192.30	272.30	41	35
Delaware	69.00	------	------	115.35	184.35	43	33	69.00	------	-----	------	192.30	261.30	43	39
Maryland	75.00	20.00	------	115.35	210.35	34	29	75.00	20.00	-----	------	192.30	287.30	35	36
District of Columbia	50.00	53.36	1.00	92.28	196.64	38	45	50.00	53.36	1.00	------	153.84	258.20	44	46
West Virginia	78.00	111.25	------	115.35	304.60	10	28	156.00	111.25	-----	22.50	192.30	482.05	11	19
Southeastern:															
Virginia	45.60	59.40	------	138.42	243.42	22	34	45.60	59.40	-----	130.00	230.76	465.76	12	13
North Carolina	144.00	64.62	------	161.49	370.11	3	3	252.00	64.62	-----	------	269.22	585.84	4	5
South Carolina	125.00	63.20	1.00	138.42	327.62	8	10	125.00	63.20	1.00	150.00	230.76	569.96	6	6
Georgia	37.50	72.54	------	161.49	271.53	17	26	75.00	72.54	-----	25.00	269.22	441.76	18	20
Florida	79.00	------	.25	161.49	240.74	24	16	225.75	------	.25	250.50	269.22	745.72	1	1
Kentucky	112.00	116.25	.75	161.49	390.49	2	8	45.00	116.25	.50	121.00	269.22	551.97	8	9
Tennessee	80.00	------	.50	161.49	241.99	25	15	140.00	------	.50	12.50	269.22	422.22	21	11
Alabama	50.00	25.46	.75	138.42	214.63	33	30	50.00	25.46	.75	126.00	230.76	432.97	19	12
Mississippi	79.00	60.49	------	138.42	277.91	15	21	157.00	60.49	-----	13.00	230.76	461.25	13	15
East Central:															
Ohio	113.00	------	------	92.28	205.28	35	24	113.00	------	-----	20.00	153.84	286.84	36	32
Indiana	50.00	57.52	------	92.28	199.80	37	46	50.00	57.52	-----	12.00	153.84	273.36	40	42
Illinois	110.00	25.23	------	69.21	204.44	36	35	110.00	25.23	-----	------	115.38	250.61	45	40
Michigan	98.75	------	------	69.21	167.96	45	37	98.75	------	-----	50.00	115.38	264.13	42	38
Wisconsin	185.00	------	------	92.28	277.28	16	7	185.00	------	-----	190.00	153.84	528.84	9	4
Minnesota	80.00	------	------	115.35	195.35	39	27	80.00	------	-----	7.50	192.30	279.80	37	33
Iowa	155.00	------	------	92.28	247.28	21	14	155.00	------	-----	5.00	153.84	313.84	30	27
Missouri	40.00	18.50	------	46.14	104.64	49	49	40.00	18.50	-----	------	76.92	135.42	49	49
Southwestern:															
Arkansas	63.00	53.05	------	149.96	266.01	18	23	63.00	53.05	-----	------	250.00	366.05	26	28
Louisiana	120.00	25.45	------	207.63	353.08	7	2	240.00	25.45	-----	10.00	346.14	621.59	2	3
Oklahoma	145.00	------	.50	149.96	295.46	12	5	145.00	------	.50	------	250.00	395.50	23	16
Texas	129.50	43.09	------	92.28	264.87	19	19	129.50	43.09	-----	11.00	153.84	337.43	29	30
West Central:															
North Dakota	73.00	------	------	92.28	165.28	46	38	73.00	------	-----	50.00	153.84	276.84	39	34
South Dakota	87.50	------	185.00	92.28	364.78	4	1	87.50	------	-----	185.00	153.84	426.34	20	10
Nebraska	120.00	95.20	------	138.42	353.62	6	12	120.00	95.20	-----	10.00	230.76	455.96	14	24
Kansas	60.00	126.29	62.00	115.35	363.64	5	17	60.00	126.29	-----	125.00	192.30	503.59	10	17
Mountain:															
Montana	75.00	79.38	------	138.42	292.80	13	22	75.00	79.38	-----	60.00	230.76	445.14	17	22
Idaho	50.00	------	------	138.42	188.42	40	31	60.00	------	-----	------	230.76	290.76	34	31
Wyoming	40.00	53.40	------	92.28	185.68	41	47	40.00	53.40	-----	106.25	153.84	353.49	28	29
Colorado	25.00	53 40	------	138.42	216.82	22	39	25.00	53.40	-----	146.00	230.76	455.16	15	14
New Mexico	67.50	------	------	161.49	228.99	28	18	33.75	------	-----	62.50	269.22	365.47	27	23
Arizona	54.85	60.00	------	115.35	230.20	27	36	54.85	60.00	-----	250.00	192.30	557.15	7	7
Utah	60.00	64.63	.50	92.28	217.41	31	41	60.00	64.63	.50	------	153.84	278.97	38	43
Nevada	25.64	233.50	142.45	103.82	505.41	1	9	25.64	233.50	-----	142.45	173.07	574.66	5	26
Pacific:															
Washington	75.00	------	28.50	149.96	253.46	20	13	75 00	------	28.50	18.00	250.00	371.50	24	18
Oregon	71 10	------	90.00	138.42	299.52	11	4	71.10	------	-----	150.00	230.76	451.86	16	8
California	56.00	------	58.00	103.82	217.82	30	20	56.00	------	58.00	325.00	173.07	612.07	3	2

As a generalization, in the States in which manufacturer's rated capacity is the basis of the registration fee schedule, truck fees tend to be relatively low in all weight groups, and particularly so for heavy vehicles. Manufacturer's rated capacities, as generally used in the past, have lost their significance and usefulness as measures of actual carrying capacity, and in so doing have also lost most of their value as bases for taxation. Most States have converted to a much more realistic gross-weight tax basis. The establishment of uniform standards of applying gross vehicle weight ratings for all trucks has been under cooperative study by State and Federal authorities and by the truck industry for some time.

Farm Service Registrations

As stated previously, special "farm" service classifications, with reduced registration fees for farmers' trucks but without restriction as to the highways they may use, are in effect in 23 States. The provision for registration of farm trucks at half of the regular fee for private trucks is a common one, but several States allow considerably larger reductions, as may be seen in tables 4 and 5. In Kentucky, for instance, the regular registration fee for the stake truck with a gross vehicle weight rating of 12,500 pounds is $32, but a farmer can register the same truck for $4.50. The flat fee of $4.50 is applicable to farm trucks with gross vehicle weight of 22,000 pounds or less, but a 22,000-pound vehicle in private use requires a registration fee of $134. Farm trucks comprised 31 percent of all trucks registered in Kentucky in 1948. In Nebraska, the 12,500-pound gross-vehicle-weight stake truck requiring a registration fee of $80 for regular private use can be registered for $12 as a farm truck. In 1948, more than 49 percent of Nebraska's trucks were registered as farm trucks.

The numbers of trucks registered in the farm classification were reported separately in 1948 by only 15 States. In 9 of these 15 States farm truck registrations accounted for more than 30 percent of the total truck registrations. It should not be forgotten, however, that the special reduced rates for registration of farm trucks are in part compensated for by the established fact that farm trucks operate fewer miles per year than other trucks. Thus the reduction on a cents-per-mile basis is not as great as it might appear from a comparison of the registration fees in tables 4 and 5.

Road-User Taxes

While the differences in registration fees between vehicles and between States are of considerable significance, one of the purposes of this study was to present the road-user taxes so that they could be considered collectively as well as individually. With State gasoline taxes ranging from 2 to 9 cents a gallon, the effect of low registration-fee schedules is often compensated by correspondingly high gasoline-tax rates, and vice versa. This is illustrated by the situation in Vermont which, in 1948, had the highest average truck registration fee in the United States—$86, compared to less than $9 in Montana—and one of the highest fees for automobiles, but

Table 7.—*Road-user and personal-property taxes on a three-axle, tractor-semitrailer combination, 40,000 pounds gross vehicle weight*

Region and State	Private							Contract, for hire							
						Rank of State								Rank of State	
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax
New England:															
Maine	$305.00	$62.50	------	$420.00	$787.50	11	11	$305.00	$62.50	------	$10.00	$540.00	$917.50	23	22
New Hampshire	240.00	74.39	------	280.00	594.39	29	31	240.00	74.39	------	2.00	360.00	676.39	37	38
Vermont	390.00	------	------	350.00	740.00	18	10	390.00	------	------	------	450.00	840.00	27	25
Massachusetts	61.00	145.99	------	210.00	416.99	45	49	61.00	145.99	------	5.00	270.00	481.99	47	48
Rhode Island	129.00	160.42	------	280.00	569.42	31	42	129.00	160.42	------	7.00	360.00	656.42	38	44
Connecticut	200.00	220.18	------	280.00	700.18	21	36	200.00	220.18	------	5.00	360.00	785.18	31	41
Middle Atlantic:															
New York	124.00	------	------	280.00	404.00	46	43	124.00	------	------	------	360.00	484.00	46	45
New Jersey	168.00	------	$2.00	210.00	380.00	48	46	168.00	------	$2.00	------	270.00	440.00	48	47
Pennsylvania	171.00	------	------	350.00	521.00	38	29	171.00	------	------	10.00	450.00	631.00	40	35
Delaware	150.00	------	------	350.00	500.00	42	33	150.00	------	------	------	450.00	600.00	43	39
Maryland	235.00	60.00	------	350.00	645.00	24	23	235.00	60.00	------	------	450.00	745.00	32	32
Dist. of Col.	100.00	95.87	2.00	280.00	477.87	43	45	100.00	95.87	2.00	------	360.00	557.87	45	46
West Virginia	61.25	206.62	------	350.00	617.87	28	41	122.50	206.62	------	30.00	450.00	809.12	30	37
Southeastern:															
Virginia	180.00	110.00	------	420.00	710.00	19	21	180.00	110.00	------	330.00	540.00	1,160.00	13	13
North Carolina	320.00	120.01	------	490.00	930.01	4	5	560.00	120.01	------	------	630.00	1,310.01	4	8
South Carolina	215.00	130.28	2.00	420.00	767.28	13	20	215.00	130.28	2.00	300.00	540.00	1,187.28	11	12
Georgia	110.00	164.92	------	490.00	764.92	15	22	225.00	164.92	------	25.00	630.00	1,044.92	17	20
Florida	192.50	------	.50	490.00	683.00	22	14	360.00	------	.50	450.50	630.00	1,441.00	1	1
Kentucky	350.00	202.50	.75	490.00	1,043.25	1	3	150.00	202.50	.50	320.00	630.00	1,303.00	5	10
Tennessee	275.00	------	.50	490.00	765.50	14	9	400.00	------	.50	17.50	630.00	1,048.00	16	14
Alabama	75.00	50.92	1.25	420.00	547.17	35	34	75.00	50.92	1.25	338.50	540.00	1,005.67	18	15
Mississippi	282.00	97.73	------	420.00	799.73	9	13	552.00	97.73	------	13.00	540.00	1,202.73	9	9
East Central:															
Ohio	220.00	------	------	280.00	500.00	41	32	220.00	------	------	30.00	360.00	610.00	42	36
Indiana	215.00	137.88	------	280.00	632.88	26	35	215.00	137.88	------	24.00	360.00	736.88	33	40
Illinois	250.00	57.82	------	210.00	517.82	40	39	250.00	57.82	------	------	270.00	577.82	44	42
Michigan	193.75	------	------	210.00	403.75	47	44	193.75	------	------	180.00	270.00	643.75	39	33
Wisconsin	240.00	------	------	280.00	520.00	39	30	240.00	------	------	590.00	360.00	1,190.00	10	7
Minnesota	360.00	------	------	350.00	710.00	20	12	360.00	------	------	7.50	450.00	817.50	29	27
Iowa	495.00	------	------	280.00	775.00	12	8	495.00	------	------	5.00	360.00	860.00	26	21
Missouri	153.00	34.94	------	140.00	327.94	49	48	153.00	34.94	------	------	180.00	367.94	49	49
Southwestern:															
Arkansas	205.00	102.69	------	455.00	762.69	17	18	205.00	102.69	------	------	585.00	892.69	25	28
Louisiana	240.00	59.38	------	630.00	929.38	5	2	480.00	59.38	------	10.00	810.00	1,359.38	3	3
Oklahoma	345.00	------	1.00	455.00	801.00	8	6	345.00	------	1.00	------	585.00	931.00	22	17
Texas	271.00	78.91	------	280.00	629.91	27	27	271.00	78.91	------	11.00	360.00	720.91	34	34
West Central:															
North Dakota	281.50	------	------	280.00	561.50	33	24	281.50	------	------	50.00	360.00	691.50	36	31
South Dakota	172.50	------	370.00	280.00	822.50	7	4	172.50	------	------	370.00	360.00	902.50	24	18
Nebraska	381.00	209.66	------	420.00	1,010.66	2	7	381.00	209.66	------	20.00	540.00	1,150.66	14	16
Kansas	130.00	234.56	74.00	350.00	788.56	10	26	130.00	234.56	------	270.00	450.00	1,084.56	15	23
Mountain:															
Montana	245.00	97.91	------	420.00	762.91	16	16	245.00	97.91	------	110.00	540.00	992.91	19	19
Idaho	125.00	------	------	420.00	545.00	36	28	160.00	------	------	------	540.00	700.00	35	30
Wyoming	80.00	99.18	------	280.00	459.18	44	47	80.00	99.18	------	400.94	360.00	940.12	21	24
Colorado	45.00	99.18	------	420.00	564.18	32	38	45.00	99.18	------	606.60	540.00	1,290.78	7	6
New Mexico	154.50	------	------	490.00	644.50	25	19	77.25	------	------	675.00	630.00	1,382.25	2	2
Arizona	115.75	111.80	------	350.00	577.55	30	37	115.75	111.80	------	500.00	450.00	1,177.55	12	11
Utah	150.00	100.78	.50	280.00	531.28	37	40	150.00	100.78	.50	------	360.00	611.28	41	43
Nevada	69.75	200.84	280.25	315.00	865.84	6	15	69.75	200.84	------	280.25	405.00	955.84	20	29
Pacific:															
Washington	156.00	------	53.25	455.00	664.25	23	17	156.00	------	53.25	36.00	585.00	830.25	28	26
Oregon	139.50	------	437.50	420.00	997.00	3	1	139.50	------	------	562.50	540.00	1,242.00	8	5
California	112.00	------	130.00	315.00	557.00	34	25	112.00	------	130.00	650.00	405.00	1,297.00	6	4

its total road-user revenue per vehicle was well down among the States as the result of a 4½-cent gasoline tax.

This leveling out in some States of registration fees and gasoline taxes, considered in combination, is further brought out in the present study. Kentucky, for instance, ranks 40th, 34th, 12th, and 5th in amounts of registration fees paid on the four commercial vehicles in private use, indicating a relatively low registration fee on the lightest truck and a gradual increase to a relatively high fee for the 40,000-pound tractor-semitrailer. When coupled with a 7-cent gasoline tax rate, the corresponding ranks of these vehicles among the States become much more uniform—12th, 17th, 8th, and 3rd, respectively. The weight of the gasoline tax payments compared with registration fees is also exemplified in the fact that the ranks by total road-user revenues are at a considerably higher level than the ranks of registration fees considered alone.

The tendency of a low gasoline tax rate to affect total road-user payments in the opposite direction is demonstrated by the fees paid in Ohio. The registration fees for the four private-use trucks rank 2nd, 9th, 11th, and 19th, indicating a high registration fee for the lightest truck and progressively lower fees, although above the national average, for the heavier vehicles. With the addition of gasoline tax payments at a 4-cent rate, total road-user revenues rank 18th, 24th, 24th, and 32nd among the States, a somewhat more uniform payment for all vehicles and a relatively lower total road-user tax payment, much nearer the median for the nation.

Other Taxes and Vehicle Types

The tables and charts presented in this article provide sufficient data to make general comparisons of the principal direct and recurring State taxes on motor vehicles, and are not intended to constitute a detailed analysis. A detailed study of all of the direct taxes that apply to motor vehicles would be an enormous undertaking. It is well to remember that there are taxes not included in this presentation that are closely associated with highway transportation and yield considerable amounts of revenue. Sales or use taxes (collected once on each vehicle) and drivers' licenses are two of the more important levied by the States, with titles and transfers also bringing in substantial amounts of revenue. Another large group of taxes omitted are those imposed by the Federal Government on gasoline, vehicles, and automotive products, which yielded nearly $1.4 billion during 1949. Also excluded are county, city, or other local registration fees and gasoline taxes, collected in some States.

It should also be remembered, as a practical matter, that the largest proportion of revenues for highways must be obtained primarily from automobiles and relatively light trucks, i. e., 1½-ton capacity or less. This group of automobiles and trucks includes approximately 98 percent of all vehicles registered and it is not possible to raise considerable amounts of money from additional taxes on the less than 2 percent that consists of the relatively heavy trucks.

(*Text continued on page 52.*)

Table 8.—*Road-user and personal-property taxes on a five-axle, tractor-semitrailer combination, contract, for hire; 64,000 pounds gross vehicle weight*

Region and State	Tractor-truck						Semitrailer					Combination						Rank of State	
	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Gasoline tax	Total	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Total	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax
New England:																			
Maine	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
New Hampshire	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Vermont	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Massachusetts	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Rhode Island	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Connecticut	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Middle Atlantic:																			
New York	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
New Jersey	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Pennsylvania	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Delaware	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Maryland	$35.00	$30.00	------	------	$1,250.00	$1,315.00	$200.00	$30.00	------	------	$230.00	$235.00	$60.00	------	------	$1,250.00	$1,545.00	21	20
District of Columbia	150.00	178.64	$1.00	------	1,000.00	1,329.64	65.00	68.44	$1.00	------	134.44	215.00	247.08	$2.00	------	1,000.00	1,464.08	23	24
West Virginia	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Southeastern:																			
Virginia	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
North Carolina	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
South Carolina	250.00	258.00	1.00	$200.00	1,500.00	2,209.00	135.00	107.50	1.00	$200.00	443.50	385.00	365.50	2.00	$400.00	1,500.00	2,652.50	8	12
Georgia	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Florida	507.50	------	.25	1,500.50	1,750.00	3,758.25	178.00	------	.25	------	178.25	685.50	------	.50	1,500.50	1,750.00	3,936.50	1	1
Kentucky	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Tennessee	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Alabama	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Mississippi	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
East Central:																			
Ohio	357.50	------	------	30.00	1,000.00	1,387.50	133.00	------	------	------	133.00	490.50	------	------	30.00	1,000.00	1,520.50	22	19
Indiana	300.00	339.72	------	24.00	1,000.00	1,663.72	------	112.95	------	------	112.95	300.00	452.67	------	24.00	1,000.00	1,776.67	19	23
Illinois	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Michigan	237.50	------	------	150.00	750.00	1,137.50	111.25	------	------	150.00	261.25	348.75	------	------	300.00	750.00	1,398.75	24	22
Wisconsin	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Minnesota	710.00	------	------	------	1,250.00	1,960.00	10.00	------	------	7.50	17.50	720.00	------	------	7.50	1,250.00	1,977.50	16	15
Iowa	735.00	------	------	------	1,000.00	1,735.00	60.00	------	------	------	60.00	795.00	------	------	5.00	1,000.00	1,800.00	18	18
Missouri	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Southwestern:																			
Arkansas	450.00	59.70	------	------	1,625.00	2,134.70	5.00	48.10	------	------	53.10	455.00	107.08	------	------	1,625.00	2,187.08	15	14
Louisiana	400.00	33.93	------	10.00	2,250.00	2,693.93	400.00	25.45	------	------	425.45	800.00	59.38	------	10.00	2,250.00	3,119.38	4	4
Oklahoma	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Texas	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
West Central:																			
North Dakota	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
South Dakota	562.50	------	------	415.00	1,000.00	1,977.50	80.00	------	------	315.00	395.00	642.50	------	------	730.00	1,000.00	2,372.50	12	8
Nebraska	560.00	461.64	------	10.00	1,500.00	2,531.64	1.00	59.70	------	10.00	70.70	561.00	521.34	------	20.00	1,500.00	2,602.34	9	13
Kansas	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----	----
Mountain:																			
Montana	300.00	198.46	------	210.00	1,500.00	2,208.46	300.00	66.16	------	------	366.16	600.00	264.62	------	210.00	1,500.00	2,574.62	11	11
Idaho	225.00	------	------	------	1,500.00	1,725.00	150.00	------	------	------	150.00	375.00	------	------	------	1,500.00	1,875.00	17	16
Wyoming	150.00	184.80	------	720.00	1,000.00	2,054.80	50.00	70.80	------	419.69	540.49	200.00	255.60	------	1,139.69	1,000.00	2,595.29	10	10
Colorado	50.00	184.80	------	------	1,500.00	1,734 80	50.00	70.80	------	1,489.50	1,610.30	100.00	255.60	------	1,489.50	1,500.00	3,345.10	3	3
New Mexico	127.50	------	------	------	1,750.00	1,877.50	44.50	------	------	1,125.00	1,169.50	172.00	------	------	1,125.00	1,750.00	3,047.00	5	5
Arizona	188.50	208.00	------	1,000.00	1,250.00	2,646.50	92.50	79.80	------	------	172.30	281.00	287.80	------	1,000.00	1,250.00	2,818.80	7	7
Utah	295.00	273.30	.50	------	1,000.00	1,568.80	105.00	96.40	------	------	201.40	400.00	369.70	.50	------	1,000.00	1,770.20	20	21
Nevada	85.50	374.22	------	400.00	1,125.00	1,984.72	40.05	143.37	------	157.95	341.37	125.55	517.59	------	557.95	1,125.00	2,326.09	14	17
Pacific:																			
Washington	355.50	------	85.50	25.00	1,625.00	2,091.00	225.50	------	33.00	22.00	280.50	581.00	------	118.50	47.00	1,625.00	2,371.50	13	9
Oregon	171.00	------	------	------	1,500.00	1,671.00	80.10	------	------	------	80.10	251.10	------	------	1,800.00	1,500.00	3,551.10	2	2
California	206.00	------	170.00	------	1,125.00	1,501.00	96.00	------	85.00	------	181.00	302.00	------	255.00	1,300.00	1,125.00	2,982.00	6	6

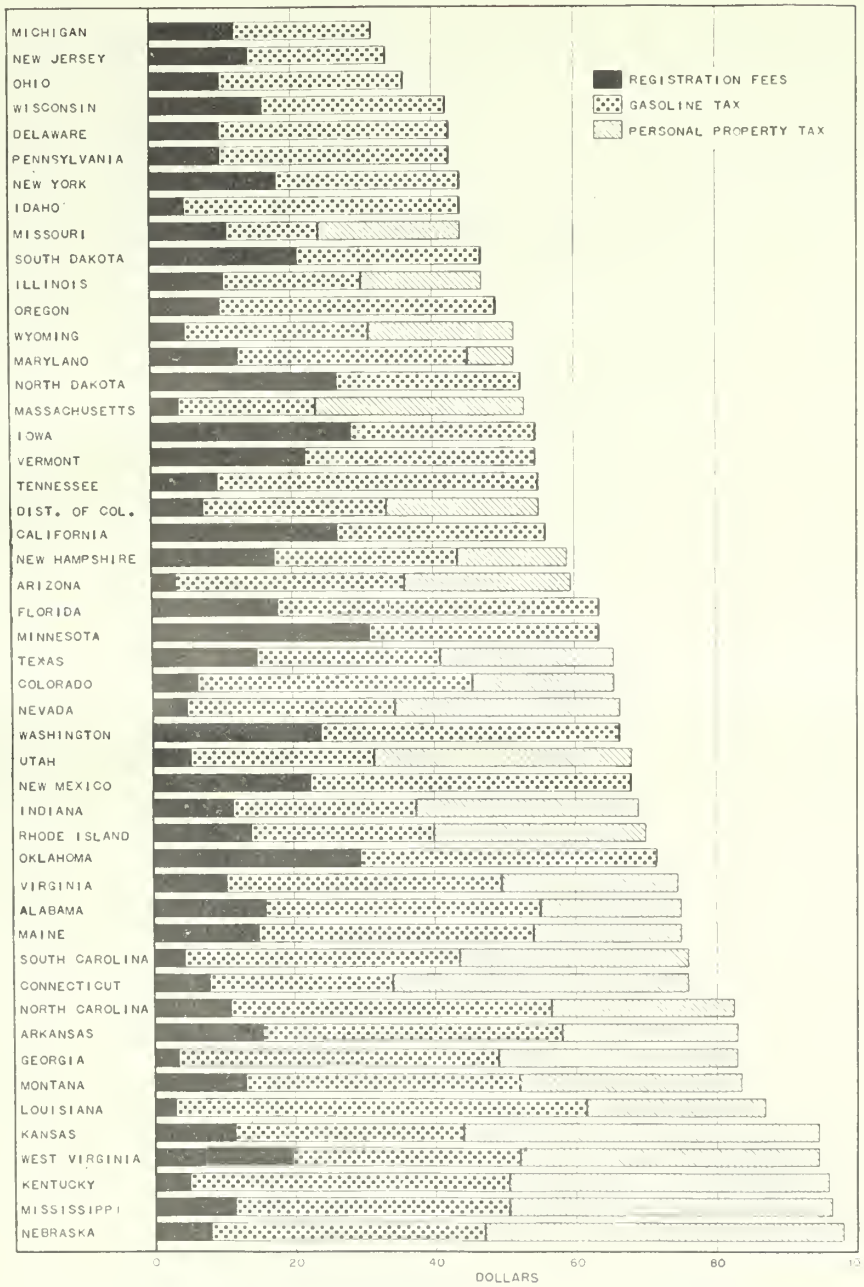

Figure 1.—Road-user and personal-property taxes on a passenger car (light- and medium-weight combined), in each State, ranked according to total tax.

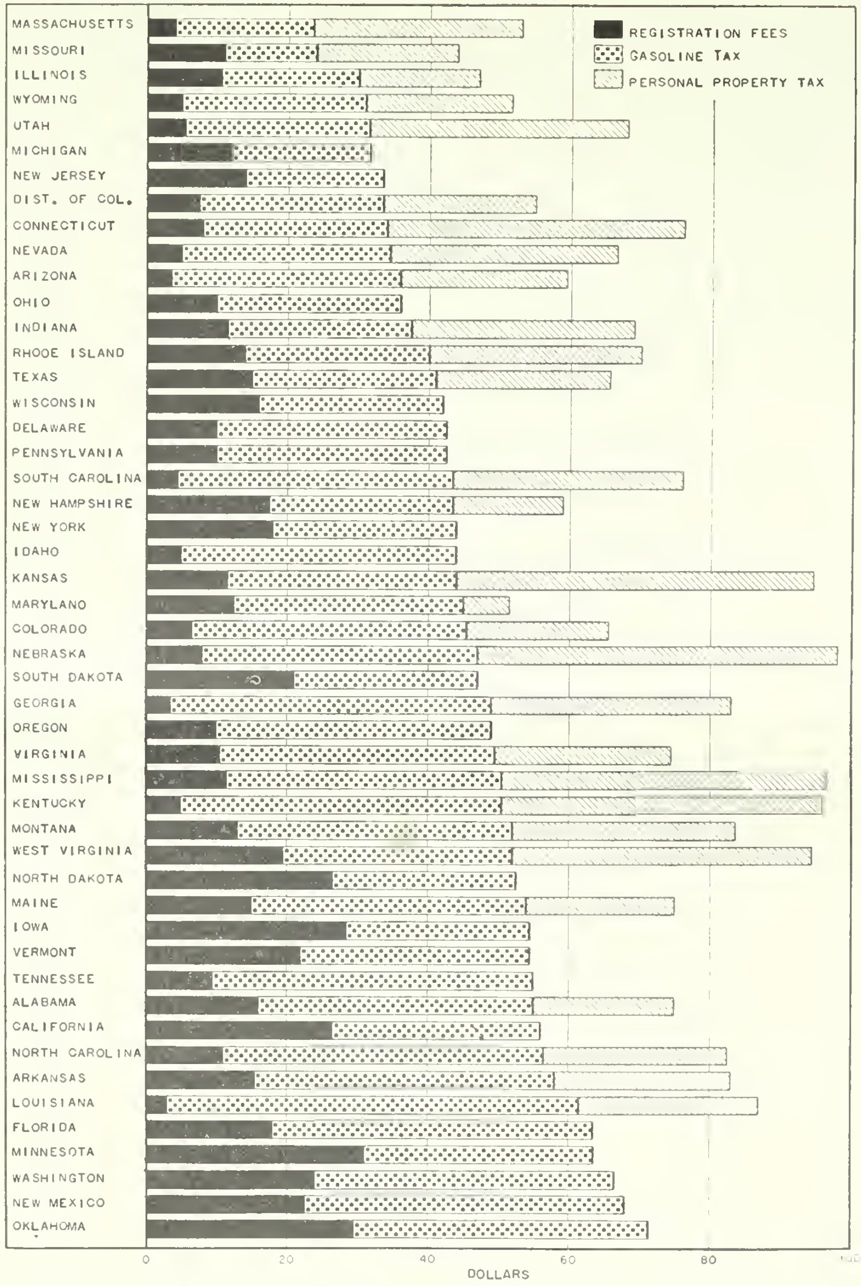

Figure 2.—Road-user and personal-property taxes on a passenger car (light- and medium-weight combined), in each State, ranked according to road-user taxes.

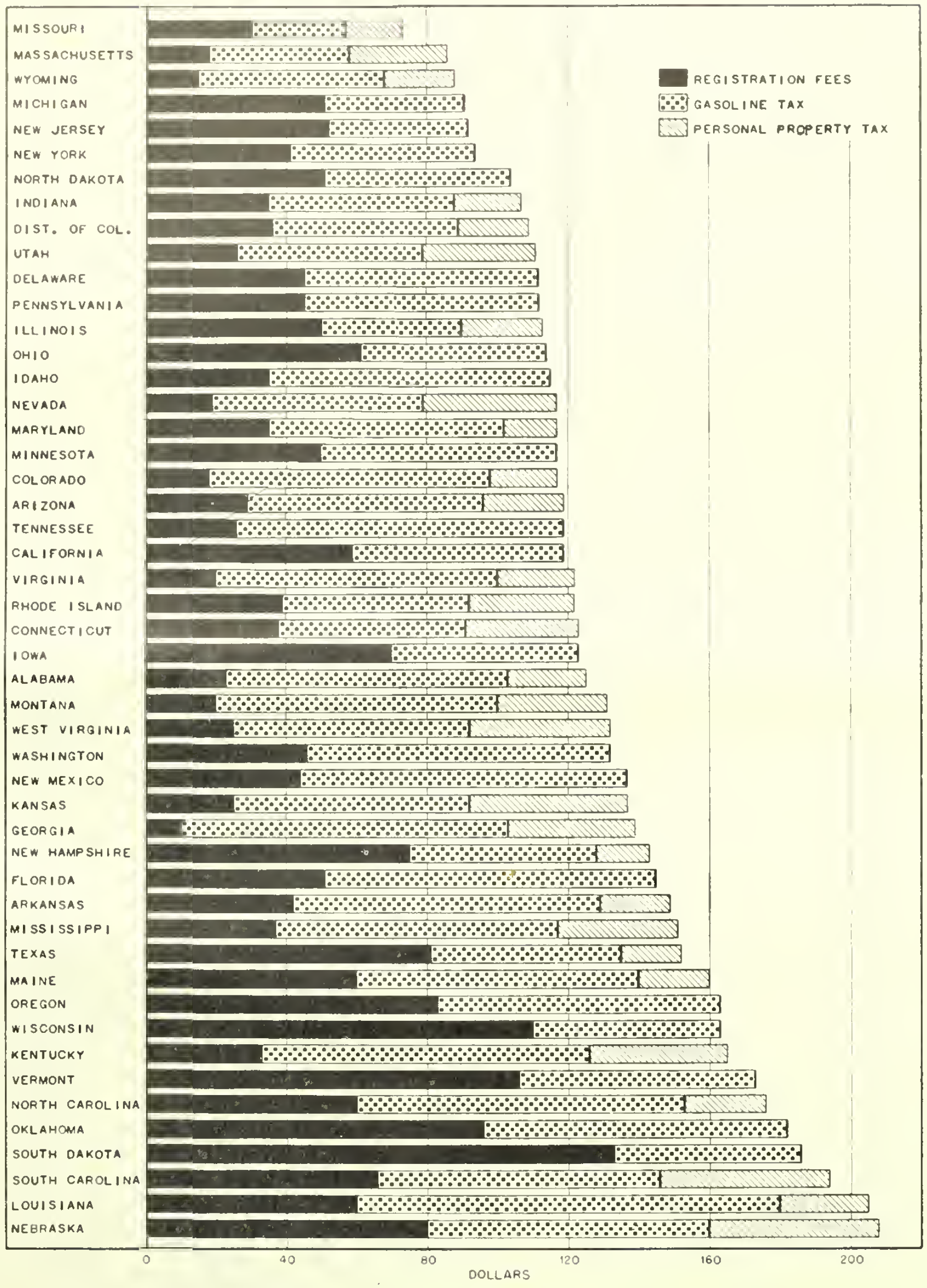

Figure 3.—Road-user and personal-property taxes on a single-unit stake truck in private use, in each State, ranked according to total tax.

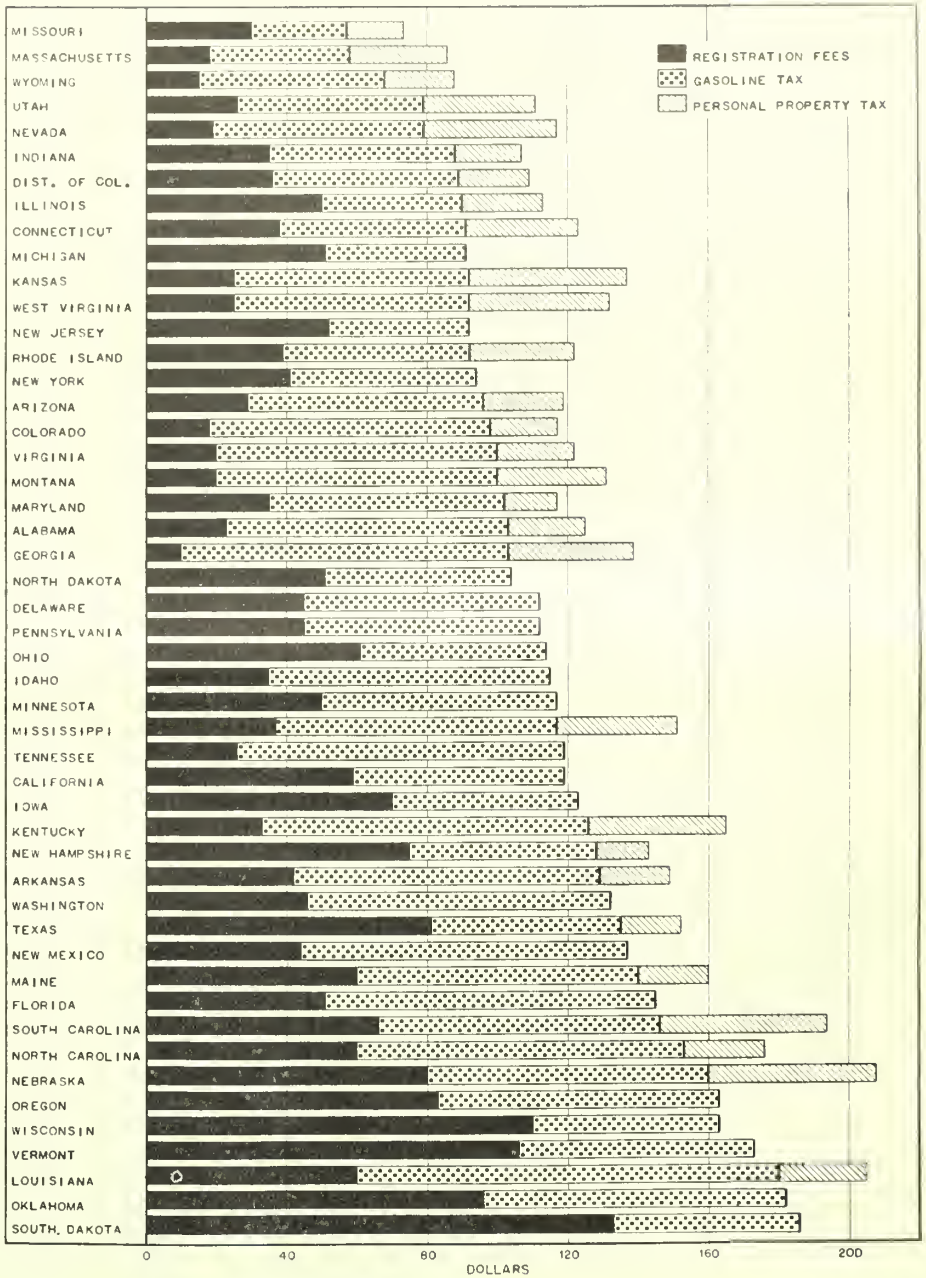

Figure 4.—Road-user and personal-property taxes on a single-unit stake truck in private use, in each State, ranked according to road-user taxes.

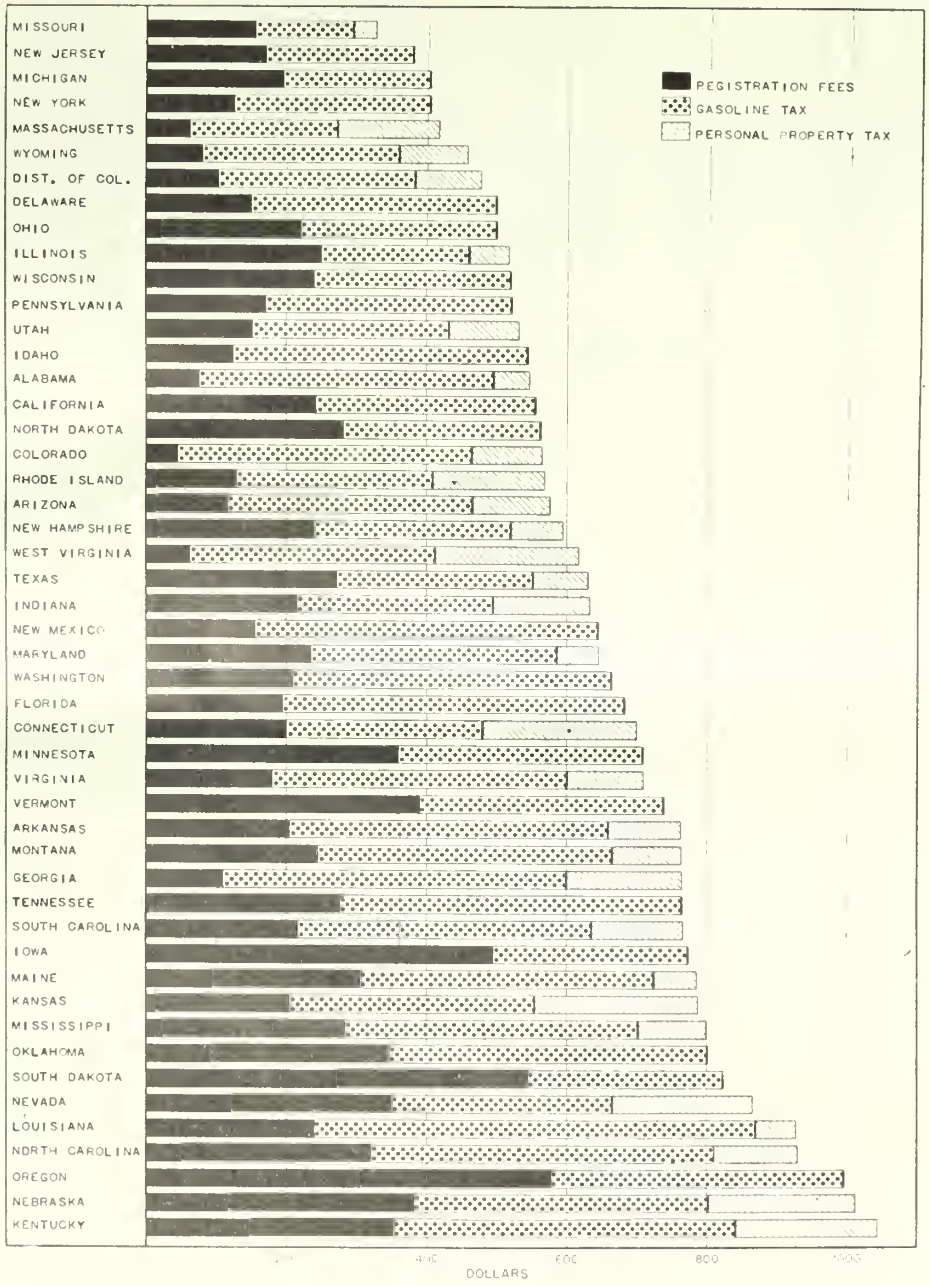

Figure 5.—Road-user and personal-property taxes on a three-axle tractor-semitrailer of 40,000 pounds gross weight, in each State, ranked according to total tax.

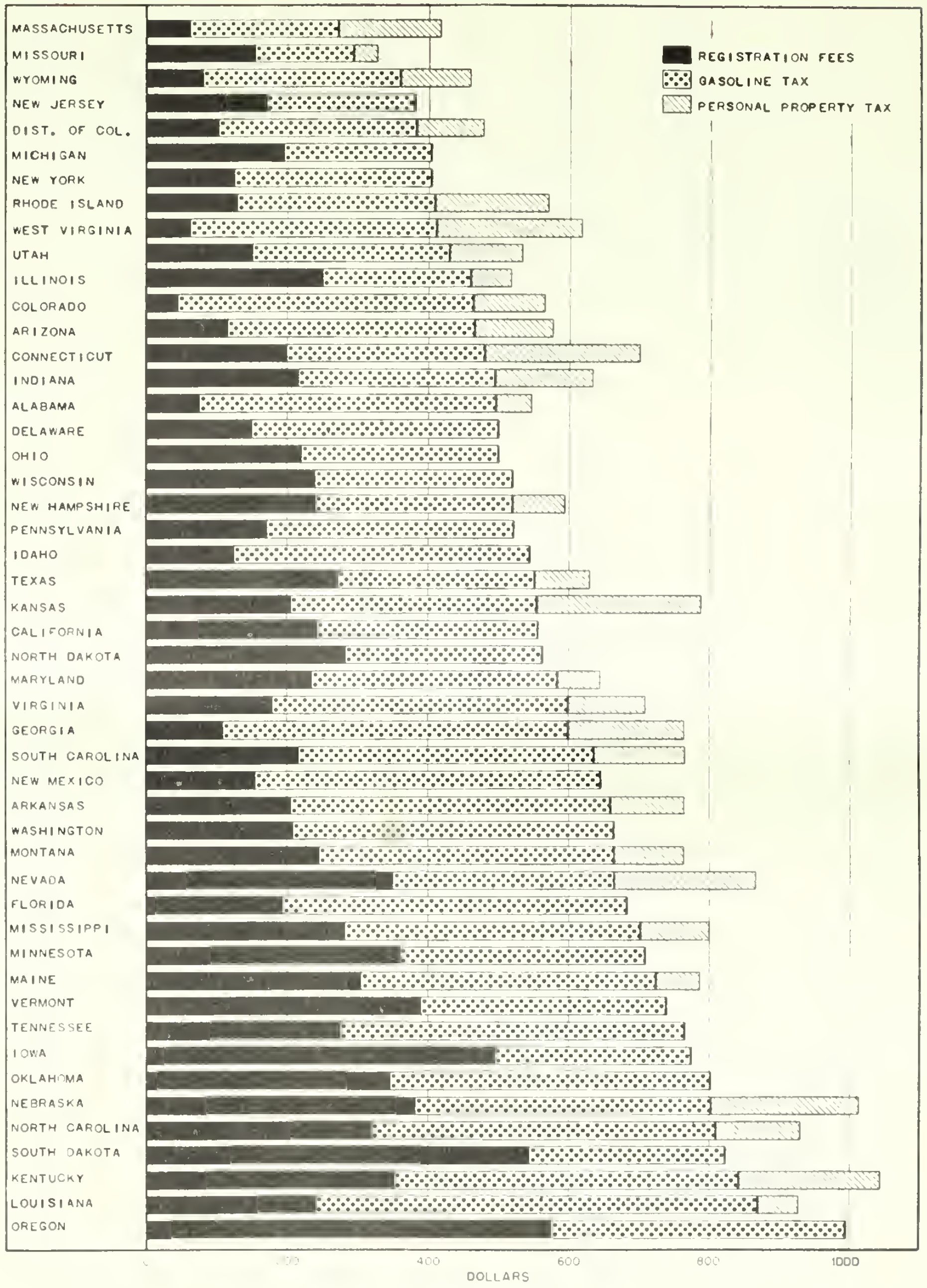

Figure 6.—Road-user and personal-property taxes on a three-axle tractor-semitrailer of 40,000 pounds gross weight, in each State, ranked according to road-user taxes.

The Identification of Rock Types

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by D. O. WOOLF,
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The interest of highway engineers in selecting the best rock for use in different types of highway construction suggests the need for a simple method of identification of rock types which can be applied in the field. A suitable method which will assist the highway engineer in identifying most of the different types of rock with which he is concerned is presented in this article. An attempt has been made to present this method in simple terms for the benefit of those who are not familiar with expressions normally used in petrographic descriptions.

THE METHOD of identification of rock presented here is intended to be applied to pieces of a size large enough so that the structure of the rock can be observed clearly. In small pieces, the alinement, if any, of the minerals composing the rock may not be observed as readily as it would be in a larger piece. The user of this method is urged to obtain for study as large a piece of the rock as is convenient to handle. Pieces about 3 inches by 4 inches by 2 inches thick may be found to be suitable.

The equipment needed consists only of a knife blade of good steel, a small magnifying glass of 6 to 10 power, and a bottle of dilute hydrochloric acid, preferably with a dropper. If the acid is not available, household vinegar will usually serve in its place. Sometimes a fragment of quartz or a quartz crystal may be found useful, but generally crystals which are large enough to test with the quartz can be identified visually.

Table 1.—*General classification of rock*

Class	Type	Family
Igneous	Intrusive (coarse-grained)	Granite [1] Syenite [1] Diorite [1] Gabbro Peridotite Pyroxenite Hornblendite
	Extrusive (fine-grained)	Obsidian Pumice Tuff Rhyolite [1] [2] Trachyte [1] [2] Andesite [1] [2] Basalt [1] Diabase
Sedimentary	Calcareous	Limestone Dolomite
	Siliceous	Shale Sandstone Chert Conglomerate [3] Breccia [3]
Metamorphic	Foliated	Gneiss Schist Amphibolite Slate
	Nonfoliated	Quartzite Marble Serpentinite

[1] Frequently occurs as a porphyritic rock.
[2] Included in general term "felsite" when constituent minerals cannot be determined quantitatively.
[3] May also be composed partially or entirely of calcareous materials.

This method follows that given by Pirrson and Knopf,[1] which in turn was based on that given by Geikie in his Textbook of Geology.[2] It uses a combination of simple physical and chemical determinations to identify the rock. In some cases the individual minerals can be identified, and the rock can be named from a knowledge of its component minerals, their abundance, and the relative amounts of each. In other cases the minerals may be too small to identify with the hand lens, and recourse must be had to the general distribution of the minerals and to the structure of the rock. It should not be expected that this method will permit the identification of any and all rocks, but it is believed that the user will be able to identify the more common rocks used in highway construction. In those cases where the rock can not positively be identified, the user of this method should have no hesitancy in stating which type of rock the sample under study most closely resembles, and describing the particular features of the sample which are not in agreement with the characteristics of the type mentioned.

General Classification

A general classification of rocks of interest in highway construction is given in table 1.[3] The rocks are first separated into three classes—igneous, sedimentary, and metamorphic—on the basis of their origin, and each class is subdivided with regard to physical characteristics or chemical composition.

In the igneous class, the intrusive or coarse-grained rocks include such familiar materials as granite and gabbro. These rocks were formed from molten material and cooled slowly so that the crystals composing the rock developed to an appreciable size. The extrusive rocks were also formed from molten material, but these cooled so rapidly that the crystals are very small. In a few cases, the molten material formed as a glass, resulting in obsidian or similar rocks. The fine-grained crystalline rocks include rhyolite, trachyte, andesite, basalt, and diabase. The first three of these rocks are sometimes grouped under a general family name of felsite which includes light- to medium-colored, very fine-grained igneous rocks. Basalt and diabase are frequently described in engineering terminology as "trap" rock.

The sedimentary class of rocks, formed by deposition of water- or wind-transported rock grains, is separated into two groups on the basis of the principal mineral component. The calcareous rocks, which are composed essentially of compounds of lime or magnesia, include limestone and dolomite. Sedimentary rocks, which are composed chiefly of silica, include shale, sandstone, and chert.

The metamorphic class is separated into two groups based on the structure of the rock. In the foliated or layered types are included gneiss, schist, and slate, while quartzite and marble are included in the nonfoliated type. The metamorphic class includes those rocks which have been formed from another type of rock by heat or pressure. For example, gneiss may be formed from granite, marble from limestone or dolomite, and quartzite from sandstone. Sometimes this alteration

Table 2.—*Rock-forming minerals*

Name	Composition
Primary minerals	
Quartz	Silicon dioxide.
Feldspar:	
Orthoclase	Silicate of potassium and aluminum.
Microcline	Silicate of potassium and aluminum.
Plagioclase	Silicate of sodium, calcium, and aluminum.
Pyroxene:	
Augite	Silicate of calcium, iron, magnesium, and aluminum.
Amphibole:	
Hornblende	Complex silicate principally of calcium, iron, magnesium, and aluminum.
Mica:	
Muscovite	Hydrous silicate of potassium and aluminum.
Biotite	Hydrous silicate of potassium, magnesium, iron, and aluminum.
Magnetite	Iron oxide.
Rock glass	Variable.
Garnet	Silicate of aluminum, iron, and calcium.
Olivine	Silicate of magnesium and iron.
Secondary minerals	
Calcite	Calcium carbonate.
Dolomite	Calcium and magnesium carbonate.
Kaolin	Hydrous silicate of aluminum.
Chlorite	Hydrous silicate of iron, magnesium, and aluminum.
Epidote	Hydrous silicate of calcium, aluminum, and iron.
Limonite	Hydrous iron oxide.
Opal	Hydrous silicon dioxide.

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[1] *Rocks and Rock Minerals*, by Louis V. Pirrson and Adolph Knopf, 3rd edition; John Wiley and Sons, Inc., New York; 1947.

[2] *Textbook of Geology*, by Sir Archibald Geikie, 3rd edition; MacMillan and Co., London; 1893.

[3] Based on table 2 in *Relation of Mineral Composition and Rock Structure to the Physical Properties of Road Materials*, by E. C. E. Lord; U. S. Department of Agriculture Bulletin No. 348, April 4, 1916.

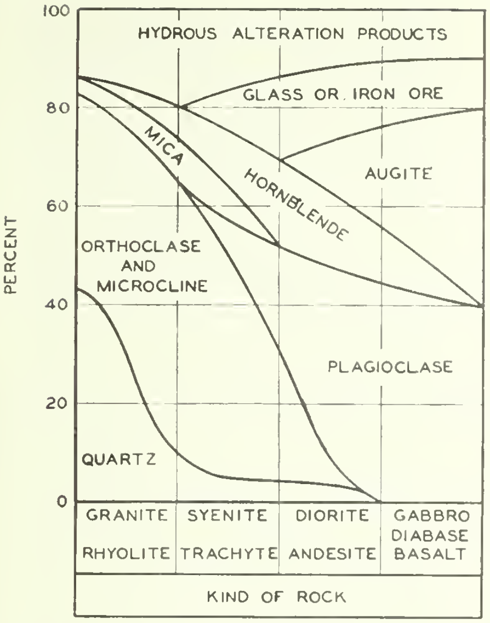

Figure 1.—Diagrammatic representation of the mineral composition of igneous rocks.

improves the quality of the rock, as in the case of quartzite, which is a much harder and tougher material than sandstone. In other cases, the reverse applies: marble generally is inferior to limestone or dolomite as an aggregate for highway construction.

Mineral Composition of Rocks

The more important rock-forming minerals are listed in table 2. These are separated into primary and secondary minerals depending upon whether they are found in the original igneous rocks or were derived by alteration of the minerals in these rocks.

The essential mineral composition of the more common rocks used in highway construction is given in table 3. This is a condensation of material given in United States Department of Agriculture Bulletin No. 348.[4] In table 3, the average percentage distribution of the minerals which are characteristic of each variety of rock is shown, together with incidental minerals which are indicated by values in parentheses. Minerals which are present in the rocks in amounts less than 3 percent are not mentioned separately but are grouped in the table under "remainder."

In figure 1, a graphical representation of the composition of igneous rocks is shown. This is based on the data given in table 3, but has been idealized to a certain extent for purposes of simplification. In the segment marked "glass or iron ore," the rock glass applies to the fine-grained extrusive rocks only.

The color of the rock furnishes some indication of the mineral content. If the rock is white or light in color, the predominant minerals probably are quartz and feldspar. Red, brown, green, gray, and black colors usually indicate the presence of minerals containing iron. In sedimentary rocks, gray or black colors may be caused by carbonaceous matter.

Classification by Geologic Type

It will be helpful if the user can classify rock with respect to its general geologic type, that is, whether the rock was formed directly from a molten mass (igneous class), or was formed by deposition of the rock grains transported by water or wind (sedimentary class), or was formed by the action of heat or pressure or both on previously existing rock (metamorphic class). Features which will assist in this classification of rock are the following:

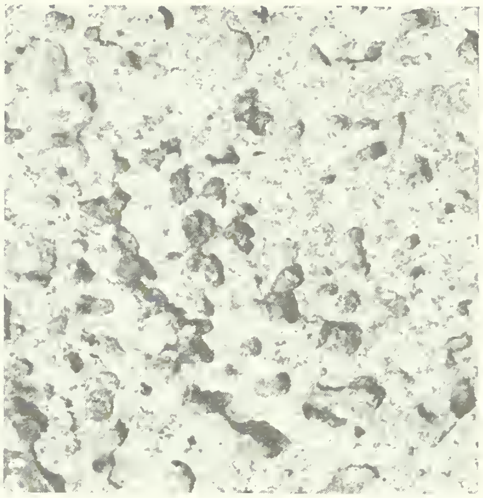

Figure 2.—A vesicular basalt (full size).

Igneous class:

- Absence of fossils.
- Presence of glass.
- Uniformity of structure.
- Interlocking crystals.

Sedimentary class:

- Rounded grains.
- Presence of fossils.
- Stratification in relatively thick layers.
- Abrupt changes in color from layer to layer.

Metamorphic class:

- Separation of crystals into approximately parallel layers.
- Formation in thin parallel layers.
- Broken readily into thin slabs.

All features mentioned for a given class probably will not be found in one single piece of rock, but one or more of those mentioned should be found.

Rock Structure

The structure of the rock is of considerable assistance in determining the general classification of the rock and also in determining the precise name for the material. Masses of rock which show a marked resemblance to columns are unquestionably of an igneous origin. Rock which is vesicular—that is, containing large or small cavities which sometimes are separated by thin walls of rock—is usually an igneous type. An example of this is shown in figure 2. In some igneous rocks these cavities are filled with a material which is of a different nature from that of the rock itself, and the rock is said to have an amygdoloidal structure.

Most types of metamorphic rocks show a peculiarity of structure which is described as foliation. Such rocks could as well be described as banded or layered, except that these terms imply an abrupt change in the appearance of the rock from one layer to the next. Bands and layers are frequently used in descriptions of sedimentary rocks. Three types of folia-

Table 3.—*Mineral composition of rocks*

Name of rock	Number of samples tested	Essential mineral composition, percent [1]													
		Quartz	Orthoclase Microcline	Plagioclase	Augite	Hornblende	Mica	Calcite	Dolomite	Chlorite	Kaolin	Epidote	Iron ore	Rock glass	Remainder
Igneous rocks:															
Granite	165	30	45	(8)	----	----	6	----	----	----	(6)	----	----	----	5
Biotite granite	51	27	41	9	----	----	11	----	----	----	(7)	----	----	----	5
Hornblende granite	20	23	34	12	----	13	4	----	----	----	(10)	----	----	----	4
Augite syenite	23	(4)	52	7	8	----	4	----	----	(3)	(11)	(3)	(4)	----	4
Diorite	75	8	7	30	----	27	(4)	----	----	(3)	(8)	(5)	(3)	----	5
Gabbro	50	----	----	44	28	9	----	----	----	(3)	(6)	----	----	----	10
Rhyolite	43	32	45	(3)	----	----	(5)	----	----	(4)	(3)	----	(4)	----	4
Trachyte	6	(3)	42	----	----	6	----	(3)	----	(3)	(14)	(8)	(7)	9	5
Andesite	67	----	----	48	14	3	----	----	----	(6)	----	(3)	(8)	12	6
Basalt	70	----	----	36	35	----	----	----	----	----	----	----	(3)	21	5
Altered basalt	196	----	----	32	31	----	----	----	----	(9)	(4)	----	(4)	12	8
Diabase	29	----	----	44	46	----	----	----	----	----	----	----	(4)	----	6
Altered diabase	231	----	----	35	26	----	----	----	----	(15)	(9)	----	(4)	----	11
Sedimentary rocks:															
Limestone	875	(6)	----	----	----	----	----	83	8	----	----	----	----	----	3
Dolomite	331	(5)	----	----	----	----	----	11	82	----	----	----	----	----	2
Sandstone	109	79	(5)	----	----	----	----	----	----	----	(4)	----	(9)	----	3
Feldspathic sandstone	191	35	26	----	----	----	----	(3)	----	(3)	(22)	----	(4)	----	7
Calcareous sandstone	53	46	(3)	----	----	----	----	42	----	----	----	----	(3)	----	6
Chert	62	93	----	----	----	----	----	----	----	----	----	----	----	----	[2] 7
Metamorphic rocks:															
Granite gneiss	107	37	35	(3)	----	----	18	----	----	----	----	----	----	----	7
Biotite gneiss	62	31	35	(5)	----	----	21	----	----	----	----	----	----	----	8
Hornblende gneiss	18	10	16	15	(3)	45	(4)	----	----	----	----	----	----	----	7
Mica schist	42	37	16	----	----	----	38	----	----	----	----	----	----	----	9
Biotite schist	17	34	13	(3)	----	----	41	----	----	----	----	----	----	----	9
Chlorite schist	23	11	----	10	----	(5)	----	----	----	39	----	28	(4)	----	3
Hornblende schist	68	10	(3)	12	----	61	----	----	----	----	----	(7)	----	----	7
Amphibolite	22	(3)	----	8	----	70	----	----	----	----	----	12	----	----	7
Slate	71	29	(4)	----	----	----	55	----	----	----	----	----	(5)	----	7
Quartzite	61	84	(3)	----	----	----	(4)	----	----	----	----	----	----	----	9
Feldspathic quartzite	22	46	27	----	----	----	(7)	----	----	(3)	(10)	----	----	----	7
Pyroxene quartzite	11	29	19	15	24	----	----	----	----	----	----	----	(5)	----	[3] 8
Marble	61	(3)	----	----	----	----	----	96	----	----	----	----	----	----	1

[1] Values shown in parentheses indicate minerals other than those essential for the classification of the rock.
[2] Includes 3 percent opal.
[3] Includes 3 percent garnet.

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[4] See footnote 3, page 44.

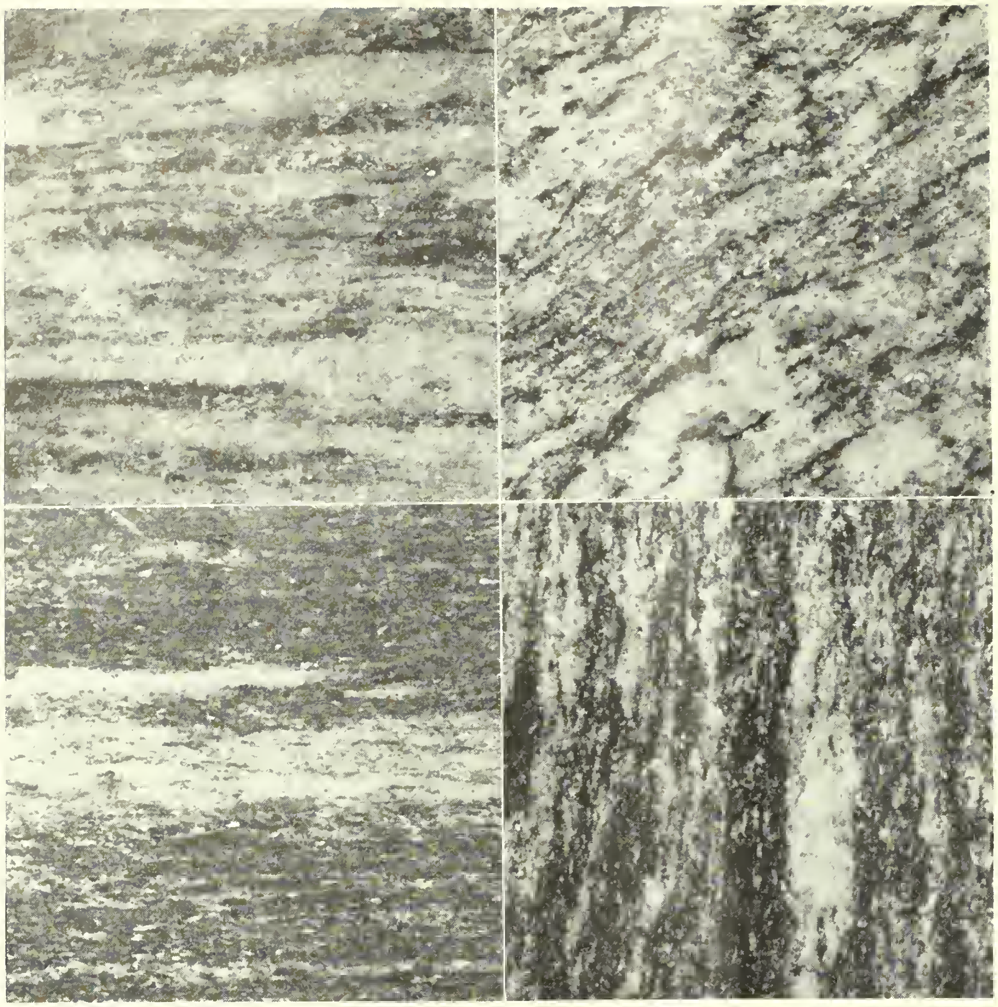

Figure 3.—Typical banding in foliated metamorphic rocks: granite gneiss (upper left); biotite gneiss (upper right); hornblende schist (lower left); mica schist (lower right).

tion—gneissoid, schistose, and slaty—are used in descriptions of metamorphic rocks. All of the foliated rocks will split or cleave more or less readily in one plane, and the type of foliation describes the degree of smoothness of the cleaved surface. Rocks with a gneissoid foliation have a rough, uneven surface while those with a slaty foliation have a very smooth cleaved surface. Schists or rocks with a schistose foliation have cleaved surfaces which are much smoother than the gneisses but not as smooth as the slates.

Under the effect of heat or pressure, the minerals in the foliated metamorphic rocks have been caused to arrange themselves in more or less parallel planes. The dark-colored minerals may separate from the light-colored minerals and form bands or streaks which are characteristic of certain foliated rocks. Figure 3 shows typical specimens of gneiss and schist. A close examination of such rocks will show that there is seldom an abrupt and complete separation between the dark- and light-colored minerals. Usually there is a zone of transition from light to dark bands, or the dark bands will contain an appreciable percentage of light-colored minerals. Banded sedimentary rocks which may be confused with the metamorphic rocks generally have an abrupt change in color or texture from one layer to the next, as shown in figure 4

Identification of Quartz and Feldspar

In the use of this method, it is necessary that quartz and feldspar be identified when these minerals occur in crystals or grains large enough to be seen with the aid of the hand lens. Their identification as constituents of rock may present some difficulties, since even in rocks of coarse grain these minerals may be so small that the usual determinations made on the ordinary mineral specimen frequently cannot be applied. For example, in hand specimens of minerals, feldspar may be identified in part by the fact that it is scratched by quartz. This test can seldom be applied to rock specimens due to the small size of the component crystals or grains. Recourse must be made to visual examination with reference to the color, shape, luster, and fracture of the grains.

In igneous rocks, quartz usually has a gray or smoky color, while feldspar is white, gray, or various shades of red. Grains of quartz are usually transparent or translucent, but those of feldspar are opaque. Quartz and feldspar are dissimilar in cleavage: if the grains are sufficiently large, those of feldspar will be found to break with flat surfaces forming an angle of about 90 degrees. By rotating the hand sample of rock so that light strikes the surface at different angles, the cleavage faces of feldspar crystals may be observed. A photograph showing the reflection of light from a cleavage face of a rather large crystal of feldspar in a specimen of granite is shown in figure 5. Quartz has no cleavage, and breaks with a conchoidal or shell-like fracture. Quartz has a glassy luster while feldspar has a luster more nearly like porcelain. Feldspar is affected by weathering, and the luster tends to become dull.

Table 4.—*Characteristics of feldspar and quartz as components of igneous rocks*

Characteristic	Feldspar	Quartz
Color	White, pink	Gray, smoky.
Transparency	Opaque	Translucent.
Luster	Porcelaneous to dull.	Glassy.
Cleavage	Good on two faces forming angles of about 90°.	None.
Form of crystal face.	Parallelopiped	Shapeless.
Multiple twinning.	Frequent	None.

In the crystallization of the rock minerals from the mass of molten rock, the feldspars crystallize before quartz, and tend to occur in crystal form while quartz develops in more or less shapeless masses. Feldspar crystals frequently are compound structures of intergrown crystals which developed simultaneously The longitudinal axes of the portions of the crystals are parallel, but the transverse axes of one crystal segment are rotated through 180 degrees from those of the adjacent segment. At the junction between crystal segments, a plane of twinning is produced. In rock-making feldspars, these planes of twinning usually are very closely spaced, and the cleavage surface of a crystal of feldspar may appear to have been ruled with fine, parallel lines, as shown in figure 6. This twinning is not found in quartz.

A summarization of the principal characteristics of feldspar and quartz as components of igneous rocks is shown in table 4.

Identification of Other Minerals

In some cases, the common forms of the ferromagnesian minerals augite, hornblende, olivine, and biotite may also be identified in

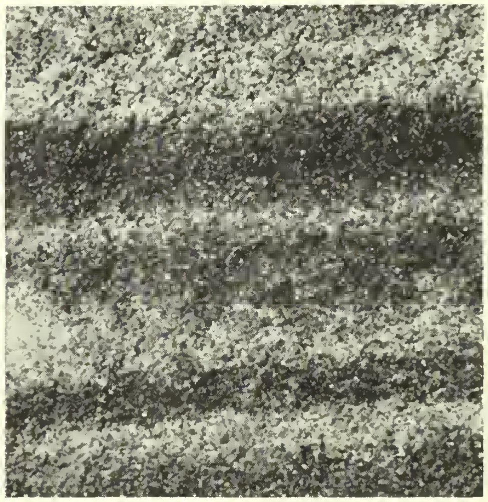

Figure 4.—Abrupt change in color between layers is characteristic of banded sedimentary rocks (banded sandstone).

rocks. The ferromagnesian minerals contain iron or magnesia or both as a principal component, and are identified by shape and color. In some rocks the minerals are sufficiently well crystallized for identification, but frequently they occur as grains or irregular masses and identification in the hand specimen may not be possible except by color.

Olivine is seldom found in well-developed crystals in rock. It occurs usually as grains or masses, and is identified by its color which varies from an olive green to a yellow green.

Augite and hornblende are the more common varieties of two large families of minerals, the pyroxenes and the amphiboles. Both augite and hornblende have a dark green to black color, and both frequently occur as grains or masses in rock. Identification of these minerals in the hand specimen depends upon whether the crystal shape can be determined. Augite tends to develop in short, thick crystals with a square or rectangular cross section. Hornblende commonly occurs as long, slender blades with irregular ends, and the cross section has a diamond shape with the acute angles replaced by parallel planes at right angles to the longer transverse axis of the crystal. Biotite is black mica, and is recognized by its black, shining color, its softness, and its occurrence as irregular flakes or scales in granites, syenites, and metamorphic rocks.

System for Rock Identification

The system for the identification of rock is shown in tabular form on page 48. In this method, all considerations are based on the appearance or character of newly fractured surfaces of the unweathered rock. In determinations of gravel, many pieces may be found to be weathered, and some modifications of the characteristics mentioned may be expected. For example, pieces of feldspar in gravel may have a dull luster, and some pieces may be so soft that they can be scratched with the knife.

The first determination in the system is the preliminary classification by which the specimen is placed in one of five general groups.

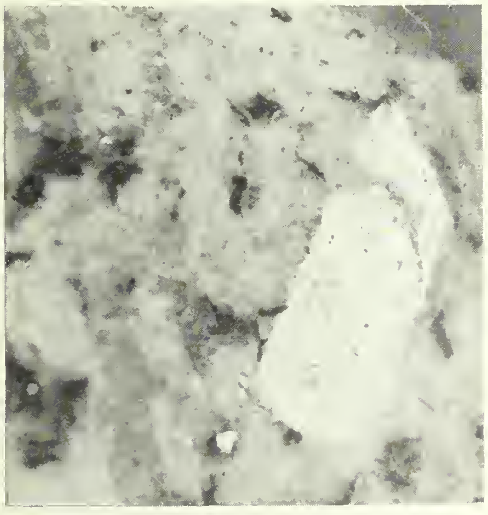

Figure 5.—Reflection of light from a cleavage face of a feldspar crystal in granite (about 2X).

When a determination of the lithological composition of a gravel is made, it is believed to be most desirable to separate the entire sample into the various general groups, and to examine each group of particles as an entity, subdividing each by the methods described. This should permit the identification to be performed most rapidly, and should group individual particles which show variations from a given class of rock due to slight differences in color, texture, or the effects of weathering.

In group I, glassy rocks, quartz is included as a rock. Actually this is not correct as quartz is a mineral, differing from rock in that it has a chemical composition expressible by a formula and also has a definite crystalline structure. However, quartz does occur in sufficiently large masses so that it is quarried as a rock, and is the most common material in a large percentage of gravels used for construction purposes. Due to these reasons, quartz is included in the table.

Subgroup II B, covering hard, fine-grained rocks, contains some types of rock which lately have become of considerable interest to those concerned with the durability of portland cement concrete. Included in the general term "felsite" are a number of varieties of rock which may be chemically reactive to a detrimental degree with the alkali in cement. These include rhyolite, trachyte, and andesite. The identification of these rocks requires that both the kind and amount of feldspar present be determined. This determination cannot be made on the hand specimen, and the practice of grouping all fine-grained, light- or medium-colored, igneous rocks containing feldspar as a major constituent under the general term of felsite has been adopted. This is believed to be a sound procedure, as the various rocks so grouped appear to have essentially the same properties from the engineering viewpoint.

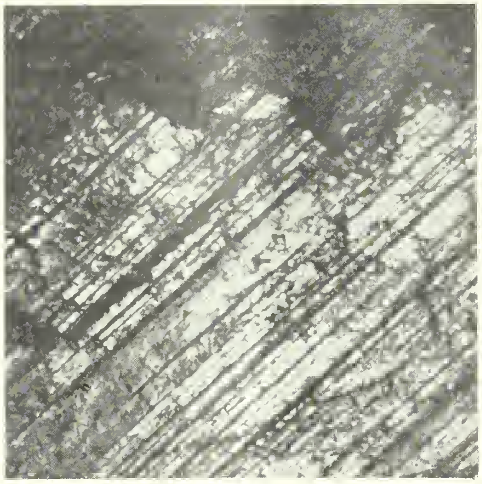

Figure 6.—Planes of twinning in feldspar produce an appearance of fine, parallel lines (about 3X).

In subgroup III B, item 5, separation between sandstone and quartzite is made by examination of the plane of fracture of the rock. If the fracture is around the grains of quartz, the rock is sandstone. If the fracture passes through the grains, or through an appreciable percentage of the grains, the rock is quartzite.

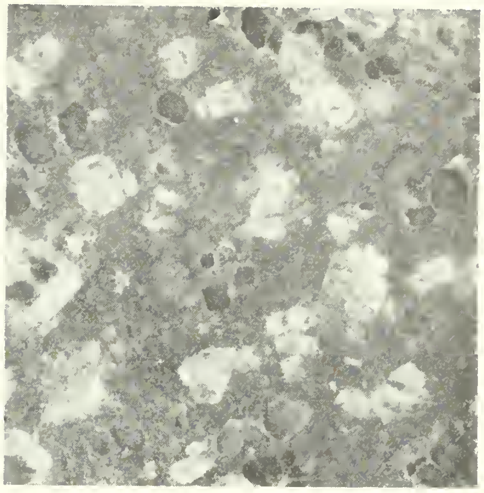

Figure 7.—Porphyritic granite (full size).

Subgroup III C covers rocks of a porphyritic texture as shown in figure 7. Originally, a porphyry referred to a rock composed of feldspar crystals embedded in a compact, dark red or purple groundmass. This name now refers to rocks containing large crystals of any kind, either well-formed or corroded to a rounded or irregular shape, which are embedded in a more finely crystalline or glassy groundmass of any color. A porphyry could refer to a rock containing crystals as big as an inch long which are embedded in a groundmass of crystals one-fourth inch in size; or it could refer to a rock containing crystals one-tenth of an inch in size embedded in a groundmass of barely visible crystals. In the laboratories of the Bureau of Public Roads, the name porphyry is used to refer to a rock containing numerous and more or less uniformly distributed crystals over one-eighth inch in size embedded in a groundmass so fine-grained that the individual grains are not recognized by the unaided eye.

Kemp [5] states that porphyries are commonly classified from the larger crystals (phenocrysts), with little regard for the composition of the groundmass even though the latter comprises over half of the rock. If the groundmass is of very fine grain, no other practice than that mentioned above can be followed without the use of the petrographic microscope or analysis by chemical methods.

In group IV, foliated rocks, it is doubted that the many varieties of schist can always be determined in the hand specimen. A few of the more common varieties are mentioned. There are many others, depending upon the presence of some mineral which may be in sufficient quantity or have certain unique properties to warrant its use as a modifier of the general term, schist. Most schists have about the same physical properties and the identification of the rock by this name alone may usually be sufficient.

In group IV, item 2(b), hornblende schists contain a small amount of quartz whereas

(*Text continued on page 32.*)

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[5] *A Handbook of Rocks*, by J. F. Kemp; 6th edition, revised by F. F. Grout; D. Van Nostrand Co. Inc., New York; 1942.

PRELIMINARY CLASSIFICATION

Group I.—Glassy, wholly or partly.
Group II.—Not glassy; dull or stony; homogeneous; so fine-grained that grains cannot be recognized.
Group III.—Distinctly granular.
Group IV.—Distinctly foliated; no effervescence with acid.
Group V.—Clearly fragmental in composition; rounded or angular pieces or grains cemented together.

GROUP I.—GLASSY ROCKS

1. Glassy luster; hard; conchoidal fracture; colorless to white or smoky gray; generally brittle. *Quartz.*
2. Solid glass; may have spherical inclusions; brilliant vitreous luster; generally black. *Obsidian.*
3. Cellular or frothy glass. *Pumice.*

GROUP II.—DULL OR STONY, VERY FINE-GRAINED ROCKS

SUBGROUP II A.—Not scratched by fingernail, but readily scratched with knife.

1. Particles almost imperceptible; dull luster; homogeneous; clay odor; little if any effervescence with acid; laminated structure; breaks into flakes. *Shale.*
2. Little if any clay odor; brisk effervescence with acid. *Limestone.*
3. Little if any clay odor; brisk effervescence with acid only when rock is powdered or acid is heated. *Dolomite.*
4. Soapy or greasy feel; translucent on thin edges; green to black; no effervescence. *Serpentinite.*

SUBGROUP II B.—Not scratched with the knife or scratched only with difficulty; no effervescence with acid.

1. Light to gray color; clay odor possible; may have a banded flow structure. *Felsite.*
2. Very hard; pale colors to black; no clay odor; conchoidal fracture; waxy or horny appearance. *Chert.* If dark gray to black, *Flint.*
3. Heavy; dark color; may have cellular structure; may contain small cavities filled with crystalline minerals. *Basalt.*

GROUP III.—GRANULAR ROCKS

SUBGROUP III A.—Easily scratched with the knife.

1. Brisk effervescence with acid. *Limestone* or *Marble.*
2. Brisk effervescence only with warm acid, or with powdered rock. *Dolomitic marble.*

SUBGROUP III B.—Hard; not scratched with knife or scratched with difficulty; grains of approximately equal size.

1. Mainly quartz and feldspar; usually light colored, sometimes pinkish. *Granite.*
2. Mainly feldspar; little quartz (less than 5 percent); light colors of nearly white to light gray or pink. *Syenite.*
3. Feldspar and a dark ferromagnesian mineral.
   (a) Major constituent feldspar; rock of medium color. *Diorite.*
   (b) Ferromagnesian mineral equal to or in excess of feldspar; rock of dark color.
      (1) Grains just large enough to be recognized by the unaided eye. *Diabase.*
      (2) Coarse-grained rock. *Gabbro.*
4. Mainly ferromagnesian minerals; generally dark green to black.
   (a) Predominant olivine with pyroxene or hornblende. *Peridotite.*
   (b) Predominant augite. *Pyroxenite.*
   (c) Predominant hornblende. *Hornblendite.*

GROUP III.—Continued

5. Mainly quartz.
   (a) Fracture around grains. *Sandstone.*
   (b) Fracture through all or through an appreciable percentage of grains. *Quartzite.*

SUBGROUP III C.—Hard; not scratched with knife or scratched with difficulty; large distinct crystals in finer groundmass.

1. Crystals of feldspar and quartz with some of a ferromagnesian mineral (generally biotite) in a light-colored groundmass (of feldspar and quartz). *Granite porphyry.*
2. Crystals of feldspar and usually a ferromagnesian mineral in a light-colored groundmass (of feldspar). *Syenite porphyry.*
3. Crystals of ferromagnesian minerals, or of striated feldspar, or both, in a medium-colored groundmass (of feldspar and ferromagnesian minerals). *Diorite porphyry.*
4. Crystals of quartz, or feldspar, or both, generally with a ferromagnesian mineral, in a predominant, fine-grained groundmass of light color. *Felsite porphyry.*
5. Crystals of feldspar, or of a ferromagnesian mineral, or both, in a fine-grained, dark or black, heavy groundmass. *Basalt porphyry.*

GROUP IV.—FOLIATED ROCKS

1. Medium to coarse grain; roughly foliated. *Gneiss.*
2. More finely grained and foliated. *Schist.*
   (a) Consists mainly or largely of mica with some quartz. *Mica schist.*
   (b) Medium green to black; consists mostly of a felted or matted mass of small, bladed or needle-like crystals arranged in one general direction. *Hornblende schist* or *amphibolite.*
   (c) Glassy or silky luster on foliation surfaces; splits readily into thin pieces. *Sericite schist.*
   (d) Soft, greasy feel; marks cloth; easily scratched with fingernail; whitish to light gray, or green. *Talc schist.*
   (e) Smooth feel; soft; glimmering luster; green to dark green. *Chlorite schist.*
3. Very fine grain; splits easily into thin slabs; usually dark gray, green, or black. *Slate.*

GROUP V.—FRAGMENTAL

1. Rounded pebbles embedded in some type of a cementing medium. *Conglomerate.*
2. Angular fragments embedded in a cementing medium. *Breccia.*
3. Fragments of volcanic (fine-grained or glassy) rocks embedded in compacted volcanic ash. *Volcanic tuff* or *Volcanic breccia.*
4. Quartz grains, rounded or angular, cemented together. *Sandstone.*
5. Quartz and feldspar grains cemented together to resemble the appearance of granite. *Arkose* (feldspathic sandstone).

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(See also adjacent column)

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1943.
1944.
1945.

MISCELLANEOUS PUBLICATIONS

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Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.

REPORTS IN COOPERATION WITH UNIVERSITY OF ILLINOIS

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No. 375 . . . Studies of Slab and Beam Highway Bridges. Part II.
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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF APRIL 30, 1950

[Thousand dollars]

State	Unprogrammed balances	Active program											
		Programmed only			Plans approved, construction not started			Construction under way			Total		
		Total cost	Federal funds	Miles	Total cost	Federal funds	Miles	Total cost	Federal funds	Miles	Total cost	Federal funds	Miles
Alabama	$14,585	$13,733	$7,031	354.9	$5,182	$2,501	161.0	$11,860	$6,161	275.3	$30,775	$15,693	791.2
Arizona	2,050	5,093	3,544	123.4	1,160	830	35.9	5,513	3,782	83.4	11,766	8,156	242.7
Arkansas	2,582	16,087	8,683	456.3	6,756	3,294	149.7	12,563	6,302	290.5	35,406	18,279	890.5
California	3,901	45,691	18,098	325.9	6,418	3,072	65.0	39,877	19,785	191.2	91,986	40,955	582.1
Colorado	5,335	4,578	2,520	178.5	1,615	1,037	46.2	15,229	8,779	285.9	21,422	12,336	510.6
Connecticut	2,809	11,315	5,442	24.7	2,681	1,410	8.8	6,718	3,843	7.3	20,714	10,695	40.8
Delaware	1,238	4,685	2,340	55.5	2,152	1,028	9.8	3,185	1,512	49.6	10,022	4,880	114.9
Florida	8,167	13,483	6,812	355.8	4,426	2,652	115.5	10,060	4,759	267.6	27,969	14,223	738.9
Georgia	6,224	16,797	8,766	558.4	5,814	3,005	135.4	38,335	18,036	629.4	60,946	29,807	1,323.2
Idaho	5,917	8,637	5,426	379.9	2,538	1,564	74.2	4,689	2,918	116.0	15,864	9,908	570.1
Illinois	26,808	43,196	22,538	367.7	12,708	6,414	167.2	45,074	21,512	286.9	100,978	50,464	821.8
Indiana	18,382	19,196	9,511	97.0	5,293	2,533	38.0	12,854	7,228	38.7	37,343	19,272	173.7
Iowa	4,310	18,414	8,212	913.4	7,230	3,204	348.5	16,039	7,934	473.7	41,683	19,350	1,735.6
Kansas	6,066	11,471	5,765	1,314.3	5,490	2,756	458.5	14,516	7,467	709.7	31,477	15,988	2,482.5
Kentucky	2,371	19,227	9,518	331.4	8,219	4,157	141.4	13,987	6,831	229.2	41,433	20,506	702.0
Louisiana	7,328	22,263	10,794	213.2	5,362	2,617	67.9	16,811	8,475	175.7	44,436	21,886	456.8
Maine	2,084	10,795	5,843	133.6	763	378	8.3	5,633	2,799	77.7	17,191	9,020	219.6
Maryland	1,802	10,032	4,795	48.9	2,016	832	14.0	15,762	7,674	53.5	27,810	13,301	116.4
Massachusetts	1,852	17,982	7,673	22.6	16,207	7,185	18.4	50,774	24,265	43.8	84,963	39,123	84.8
Michigan	11,456	16,037	8,321	618.0	7,995	4,008	131.8	38,682	15,593	173.1	62,714	27,922	922.9
Minnesota	2,381	20,209	10,513	1,753.6	9,181	4,832	506.9	14,826	7,860	143.3	44,216	23,205	2,403.8
Mississippi	15,066	461	235	24.8	1,470	731	40.0	7,179	3,740	190.6	9,110	4,706	255.4
Missouri	7,215	35,994	18,663	1,029.1	14,569	6,073	299.2	23,695	12,474	511.6	74,258	37,210	1,839.9
Montana	6,050	18,417	9,874	752.9	4,496	2,605	132.9	11,168	6,987	255.8	34,081	19,466	1,141.6
Nebraska	7,102	17,503	9,097	653.3	6,286	3,158	172.9	10,459	6,160	209.3	34,248	18,415	1,035.5
Nevada	3,046	4,939	4,071	125.5	332	271	11.6	4,698	3,847	144.5	9,969	8,189	281.6
New Hampshire	2,396	5,537	2,754	53.7	1,645	806	14.0	2,009	951	11.4	9,191	4,511	79.1
New Jersey	2,293	14,054	6,927	23.2	3,618	1,809	3.5	18,667	8,912	30.4	36,339	17,648	57.1
New Mexico	1,930	11,110	7,087	397.4	2,129	1,386	116.4	8,947	5,805	274.4	22,186	14,278	788.2
New York	34,277	81,708	42,897	290.2	22,756	11,443	68.4	86,876	43,337	155.1	191,340	97,677	513.7
North Carolina	6,448	21,839	10,679	628.8	5,089	2,411	154.7	18,893	9,103	431.5	45,821	22,193	1,215.0
North Dakota	4,061	10,238	5,268	1,576.0	6,155	3,154	342.3	6,457	3,401	473.0	22,850	11,823	2,391.3
Ohio	7,712	70,663	34,255	393.3	15,939	7,620	89.4	31,883	15,727	149.9	118,485	57,602	632.6
Oklahoma	1,533	28,194	14,963	548.1	9,266	4,494	265.1	13,506	6,657	329.1	50,966	26,114	1,142.3
Oregon	1,141	9,303	5,415	160.7	4,289	2,509	96.1	10,650	5,721	120.1	24,242	13,645	376.9
Pennsylvania	18,904	20,048	9,024	56.3	16,277	8,359	50.2	72,649	35,597	177.6	108,974	52,980	284.1
Rhode Island	218	9,777	4,976	67.0	1,680	816	1.8	9,574	4,871	5.1	21,031	10,663	73.9
South Carolina	4,959	6,460	3,247	253.3	2,854	1,475	145.6	11,777	5,906	502.3	21,091	10,628	901.2
South Dakota	2,849	12,615	7,323	1,163.7	4,285	2,696	460.8	5,396	3,277	349.6	22,296	13,296	1,974.1
Tennessee	957	23,281	11,285	619.2	7,515	3,496	161.5	15,278	7,096	276.4	46,074	21,877	1,057.1
Texas	13,027	6,552	3,477	524.9	12,862	6,330	686.5	69,011	27,976	1,728.8	79,425	37,783	2,940.2
Utah	2,530	4,679	3,435	153.1	1,867	1,376	84.8	5,472	1,001	112.6	12,018	8,812	350.5
Vermont	1,285	4,117	2,243	41.7	287	142	2.6	4,110	1,884	40.2	8,514	4,269	84.5
Virginia	9,821	20,240	10,255	541.2	5,150	2,442	151.8	14,383	7,138	230.8	39,773	19,835	923.8
Washington	2,387	15,361	7,263	154.8	3,157	1,616	67.5	16,403	7,245	113.0	34,921	16,124	335.3
West Virginia	2,699	16,297	6,904	176.5	3,218	1,657	46.1	8,988	4,448	51.1	28,503	13,009	273.7
Wisconsin	10,839	25,436	12,866	597.0	5,424	2,539	114.4	12,214	6,054	289.5	43,074	21,459	1,000.9
Wyoming	817	4,319	2,645	121.3	2,436	1,558	120.6	7,608	5,013	259.9	14,363	9,216	501.8
Hawaii	1,642	7,945	3,620	31.2	2,395	1,076	6.9	4,482	2,230	16.8	14,822	6,926	54.9
District of Columbia	1,631	5,770	3,097	8.0	-	-	-	1,210	577	.9	6,980	3,674	8.9
Puerto Rico	5,147	7,240	3,289	29.0	3,162	1,454	10.9	9,444	3,785	50.5	19,846	8,528	90.4
Total	317,630	869,018	439,279	19,816.2	289,814	144,811	6,620.9	907,073	453,435	12,093.3	2,065,905	1,037,525	38,536.4

37.8: 26/3

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AUGUST 1950

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

The earth-resistivity (upper) and refraction seismic equipment (lower) used in geophysical methods of subsurface exploration

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BUREAU OF PUBLIC ROADS

U. S. DEPARTMENT OF COMMERCE

E. A. STROMBERG, Editor

Geophysical Methods of Subsurface Exploration in Highway Construction

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported [1] by R. WOODWARD MOORE, Highway Engineer

Geophysical methods of subsurface exploration have been employed in various fields of engineering for more than 20 years. Since 1933 the Bureau of Public Roads has been studying the application of these methods to highway work, and has found that earth-resistivity and refraction seismic tests are well adapted to road construction problems. The Bureau has designed and built portable equipment for both types of test, which were described in PUBLIC ROADS in 1935.

Subsequent work, carried on during the past 15 years in 21 States, has established both methods as useful, rapid, and economical means of obtaining preliminary information on the depth and nature of subsurface formations. In this article are presented a review of the theory and method of operation of the two types of equipment, and the results of a number of actual field surveys made with them.

It will be evident from these data that, while both geophysical exploration methods are useful, the earth-resistivity test is more universally applicable to a variety of highway construction problems than the refraction seismic test. Detailed subsurface surveys can best be made by initial use of the resistivity equipment, followed by check tests with the seismic apparatus where needed. Since the fundamental principles of the two methods differ widely, concordant data from both may be accepted with considerable assurance.

SINCE 1933 the Bureau of Public Roads has had in progress a study of geophysical methods of subsurface exploration as applied to highway engineering problems, including the development of instruments and of methods of interpretation of the data obtained. Early progress was reported in papers published in 1935 (*15*) [2] and in 1936 (*17*). Both earth-resistivity and refraction seismic apparatus were adapted or developed for use in the shallow subsurface explorations usually associated with highway construction. Special attention was given to the necessity for portable units capable of being transported by hand into areas where reconnaissance surveys might be required. The cover illustrations show the earth-resistivity apparatus and the seismic equipment now in use.

A large amount of data has been obtained by the Bureau of Public Roads with this equipment applied to such problems as slope design, classification of excavation materials on grading projects, foundation studies for bridges, buildings, and other structures, investigation of tunnel sites, location of sand, gravel, solid rock, and special soils for use in construction, determination of depth of peat and muck in swampy areas, and studies of existing and potential slide areas. These field studies have been carried out in 21 States.[3]

Summary

Despite limitations that are enumerated in this article, and others which may arise in future exploration work, the geophysical methods of test have definitely established their value in connection with highway work, particularly for use in preliminary surveys. Their use by the Bureau of Public Roads and other Federal and State agencies has emphasized the value of these relatively inexpensive methods of shallow subsurface exploration in obtaining information to be used for design purposes or as control for more detailed subsurface surveys by core drilling and other commonly used direct methods. The fundamental principles of the two methods differ so widely that where both methods give concordant data they may be accepted with considerable assurance. When they are used jointly at a given location, a limited amount of confirming data from seismic tests can serve as a valuable check on a considerable number of the more inexpensive resistivity tests, at times obviating the need for test pits or auger holes for locating and identifying subsurface formations. This does not imply that test pits or auger holes may not be necessary for obtaining samples of soil and other materials for determination of their physical and other properties.

Even though there might exist some uncertainty that the geophysical methods of test would prove applicable to a particular subsurface condition, the simplicity, low cost, and rapidity with which the tests can be made recommend their trial before resorting to the more costly and tedious methods of direct exploration ofttimes employed.

Present Use

World War II caused curtailment of the use of the geophysical methods of exploration, with the general decrease in civilian construction, but an increased interest is being manifested at the present time. The New York State Department of Public Works has purchased equipment of both types and has assigned personnel to a continuing program of geophysical tests, since early in 1948, as a part of a regularly instituted program of subsurface exploration. The Pennsylvania Turnpike Commission has kept two earth-resistivity parties in the field since July 1948 in a systematic resistivity survey of well over 100 miles of right-of-way for extensions to the present Turnpike. The Michigan State Highway Department has purchased resistivity apparatus for use in locating construction materials and on other construction and maintenance problems. The Massachusetts State Department of Public Works has had in progress since 1944 a program involving the use of refraction seismic tests in studies of highway grading projects and structure sites (*29*). Wisconsin, Minnesota, Missouri, California, Texas, and Illinois have each had some experience in the application of earth-resistivity tests to highway construction problems (*5, 9, 10, 14*). The State highway departments of Georgia and Arkansas have expressed an active interest in an early application of earth-resistivity tests to their construction problems.

As a result of demonstration work done in New York, Connecticut, and New Hampshire, the Corps of Engineers of the Department of the Army adopted the seismic test as a more or less standard procedure in preliminary subsurface explorations in connection with investigations of possible dam sites for flood control. Hundreds of dam sites have been investigated by this method since the latter part of 1938 (*19, 21, 23*).

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[1] Presented at the twenty-ninth annual meeting of the Highway Research Board in Washington, D. C., December 1949.

[2] Italic numbers in parentheses refer to the chronologically arranged bibliography, p. 64.

[3] Arkansas, California, Colorado, Connecticut, Florida, Georgia, Idaho, Iowa, Maryland, Michigan, Missouri, Montana, New Hampshire, New Jersey, New York, North Carolina, Oregon, Pennsylvania, Tennessee, Virginia, Washington, and the District of Columbia.

With this brief summary of the present status of geophysics as an integral part of our highway construction program, it may be of interest to review briefly the theoretical aspects of the two methods of test and to consider in more detail their application in the field.

Theory of Refraction Seismic Method

The seismic method of subsurface exploration[4] consists of creating sound or vibration waves within the earth, usually by exploding small charges of dynamite buried 3 or 4 feet beneath the surface, and measuring the time of travel of these waves from their point of origin to each of several detectors placed at known distances from the source. The variations in mechanical energy transmitted to the detectors are converted into variations in electrical energy which, in turn, are used to deflect light rays reflected from small mirrors that are parts of sensitive galvanometers, and these deflections are recorded photographically on rapidly moving film. Electrical circuits are so arranged as to obtain one impulse at the instant of firing the explosion shot and another as the first wave reaches each detector. Figure 1 shows three typical seismic records, the small break in the right-hand trace on each film indicating the start of the wave and the three separate breaks in the three traces on each of the films indicating the arrival of the wave front at each detector. Each space between the transverse lines on the film corresponds to a time interval of 0.005 second. It is usually possible to estimate to one-tenth part of this time interval.

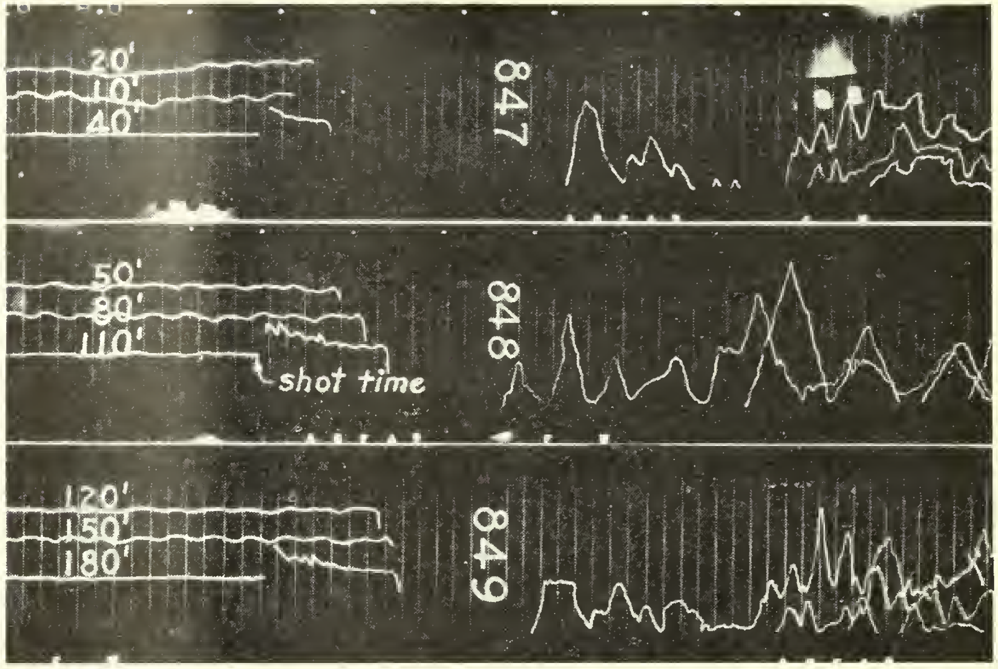
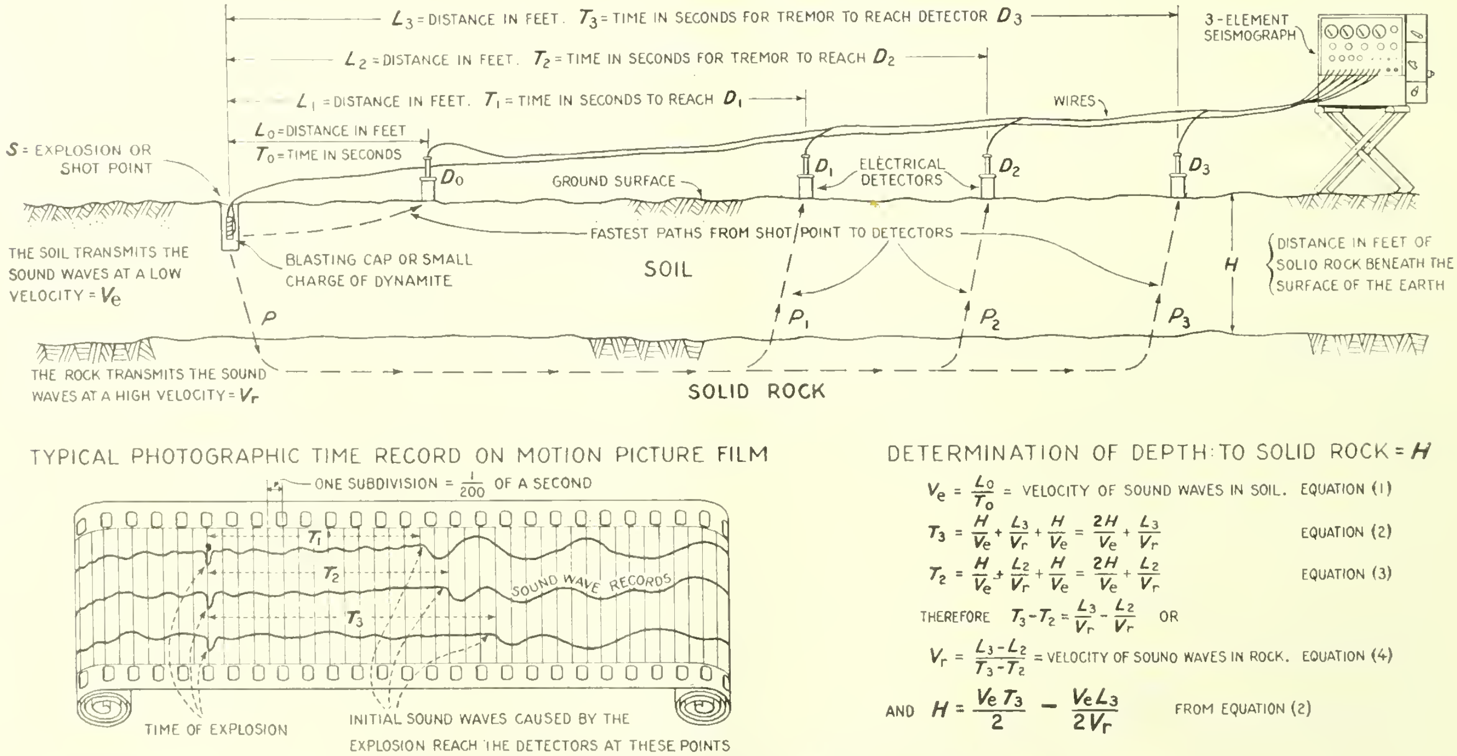


Figure 1.—Typical seismic film records.

The time lines are registered on the film by means of a suitably placed light source and a tuning fork operating at 100 cycles per second. Each tine of the tuning fork is equipped with thin phosphor-bronze plates having narrow slots which permit 200 flashes of light to reach the film during each second of time.

The time data obtained from film records and the measured distances along the ground surface between the shot point and the detectors are plotted in the form of time-distance graphs. From these the depth and probable character of the various subsurface formations are determined. Wave velocities range from approximately 600 feet per second in light loose soils to about 18,000 to 20,000 feet per second in dense, solid rock. This wide range in wave velocities makes possible a determination of the general character of the materials encountered, and by use of simple formulas the average depth to the various substrata can be calculated. A knowledge of the local geology helps materially in a more accurate identification of the formations encountered.



L_3 = DISTANCE IN FEET. T_3 = TIME IN SECONDS FOR TREMOR TO REACH DETECTOR D_3
L_2 = DISTANCE IN FEET. T_2 = TIME IN SECONDS FOR TREMOR TO REACH D_2
L_1 = DISTANCE IN FEET. T_1 = TIME IN SECONDS TO REACH D_1
L_0 = DISTANCE IN FEET
T_0 = TIME IN SECONDS
3-ELEMENT SEISMOGRAPH
WIRES
S = EXPLOSION OR SHOT POINT
D_0
D_1
ELECTRICAL DETECTORS
D_2
D_3
GROUND SURFACE
FASTEST PATHS FROM SHOT POINT TO DETECTORS
THE SOIL TRANSMITS THE SOUND WAVES AT A LOW VELOCITY = V_e
BLASTING CAP OR SMALL CHARGE OF DYNAMITE
SOIL
H
DISTANCE IN FEET OF SOLID ROCK BENEATH THE SURFACE OF THE EARTH
P
P_1
P_2
P_3
THE ROCK TRANSMITS THE SOUND WAVES AT A HIGH VELOCITY = V_r
SOLID ROCK
TYPICAL PHOTOGRAPHIC TIME RECORD ON MOTION PICTURE FILM
ONE SUBDIVISION = $\frac{1}{200}$ OF A SECOND
T_1
T_2
T_3
SOUND WAVE RECORDS
TIME OF EXPLOSION
INITIAL SOUND WAVES CAUSED BY THE EXPLOSION REACH THE DETECTORS AT THESE POINTS
DETERMINATION OF DEPTH TO SOLID ROCK = H
$V_e = \frac{L_0}{T_0}$ = VELOCITY OF SOUND WAVES IN SOIL. EQUATION (1)
$T_3 = \frac{H}{V_e} + \frac{L_3}{V_r} + \frac{H}{V_e} = \frac{2H}{V_e} + \frac{L_3}{V_r}$ EQUATION (2)
$T_2 = \frac{H}{V_e} + \frac{L_2}{V_r} + \frac{H}{V_e} = \frac{2H}{V_e} + \frac{L_2}{V_r}$ EQUATION (3)
THEREFORE $T_3 - T_2 = \frac{L_3}{V_r} - \frac{L_2}{V_r}$ OR
$V_r = \frac{L_3 - L_2}{T_3 - T_2}$ = VELOCITY OF SOUND WAVES IN ROCK. EQUATION (4)
AND $H = \frac{V_e T_3}{2} - \frac{V_e L_3}{2 V_r}$ FROM EQUATION (2)

Figure 2.—Fundamental principles of the seismograph method.

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[4] For a more detailed description of the apparatus see reference (*15*). For additional discussion of the interpretation of refraction seismic data, together with their application to various field problems, see references (*19, 21, 23*).

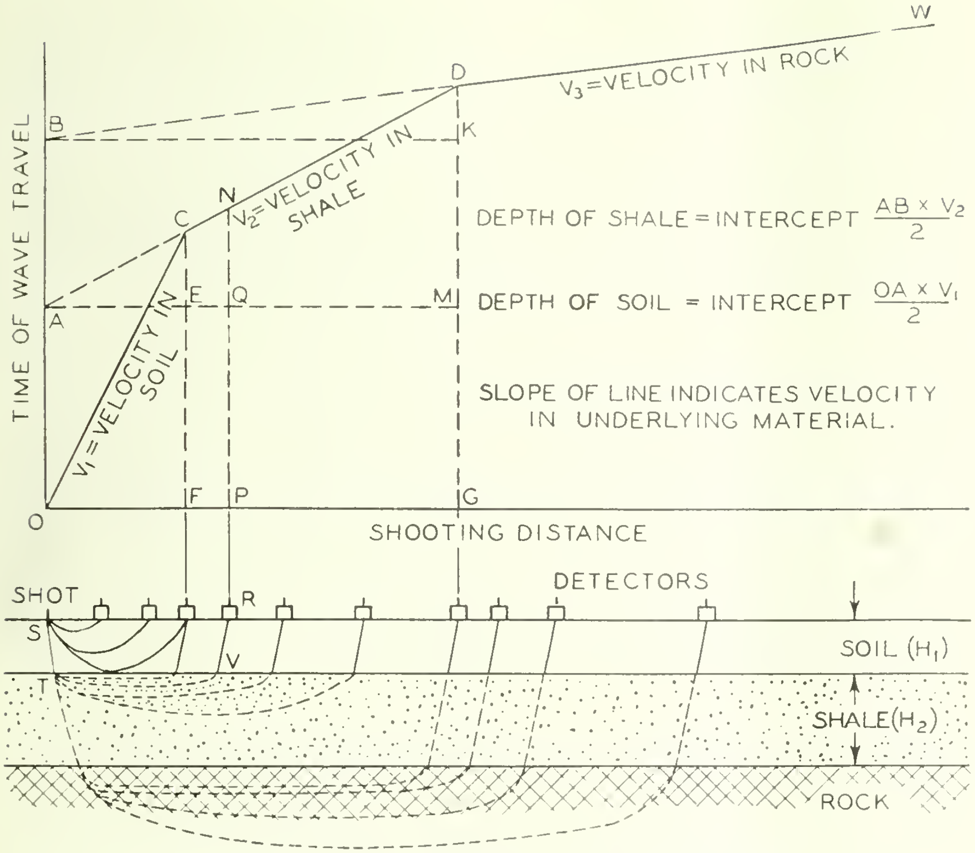


Figure 3.—Time-distance curve from which soil profile determinations are made.

Explanation of theory

The theory of refraction shooting and the derivation of approximate working formulas for depth determinations are shown in figure 2. The equations are developed on the assumption that the path of the seismic wave is vertically downward from the shot point to the rock or other dense stratum, thence along the rock to a point directly beneath the detector, and thence vertically upward to the detector. Although this assumption gives satisfactory values for the shallow depths involved in most highway problems, it is preferable to use a more exact formula for tests to greater depths such as are encountered in exploring locations for dams and certain other structures. The derivation and application of these formulas may be found in published papers (*18, 19*).

Although four detectors are shown in figure 2 to illustrate wave travel for the short distances involving the low-velocity soil and for the longer distances in the rock stratum, only three detectors are required for the three-channel seismograph used by the Bureau of Public Roads. The usual procedure, when using this type of equipment, is to place the three detectors on the ground in a line and at intervals of 25 to 50 feet apart. Shots are then fired successively at increasing distances along the extension of the line of detectors, beginning with a point 10 or 15 feet from the center detector and extending the shooting distance by increasing increments—for example, 50, 85, 125, 165, 225, and 300 feet from the center detector. There is an approximate relation between shot distance and the effective depth of the test such that this depth is about equal to one-third the shot distance. The relation depends somewhat on the relative wave velocities in the materials involved. If the depth to rock were more than about 80 feet, additional shot distances greater than the 300 feet mentioned above would be required to show a rock formation adequately. A duplicate line of shots is usually placed in the opposite direction from the center detector, expanding the data to allow depth determinations to be made when the interface between the overburden and the rock is not parallel to the surface but on a slope.

Time-distance curve

A theoretical time-distance curve is illustrated in figure 3. As shown, a straight line through the origin will result so long as a uniform homogeneous material comprises the surface layer. The velocity of wave propagation is constant in such a medium and time of wave travel is proportional to travel distance. The reciprocal of the slope of the line *OC*, passing through the origin, represents the velocity in the medium, V_1, since velocity is equal to distance divided by time.

If, at some greater depth, a second layer of homogeneous material of greater density is present, such as that designated as shale, there will be a point *F* at which there is a simultaneous arrival of a relatively slow wave through the less dense surface soil and one traveling over the longer but faster route along the top of the shale stratum. Beyond this critical distance *OF* a new slope *CD* exists, the reciprocal of which represents the faster wave travel in the shale, V_2; and for a path *STVR* the time *PQ* or *OA* is that required for the wave to travel through the surface soil from *S* to *T* and again from *V* to *R*. *QN* represents the time of travel from *T* to *V* in the shale. If H_1 is the thickness of the surface soil, we have the relation

$$H_1=\frac{V_1 \times OA}{2}$$

Similarly, for a third layer having an even greater density, such as that designated as rock, there will be a second critical distance *OG*, and a second break in the curve to a new slope *DW*, the reciprocal of which will give the velocity V_3 in the rock. The time intercept *MK* or *AB* in this instance represents the time required for the wave to travel down through the shale and back again. If H_2 is the thickness of the shale then

$$H_2=\frac{V_2 \times AB}{2}$$

In plotting the time-distance data the time units of 1/200 second, as taken directly from the

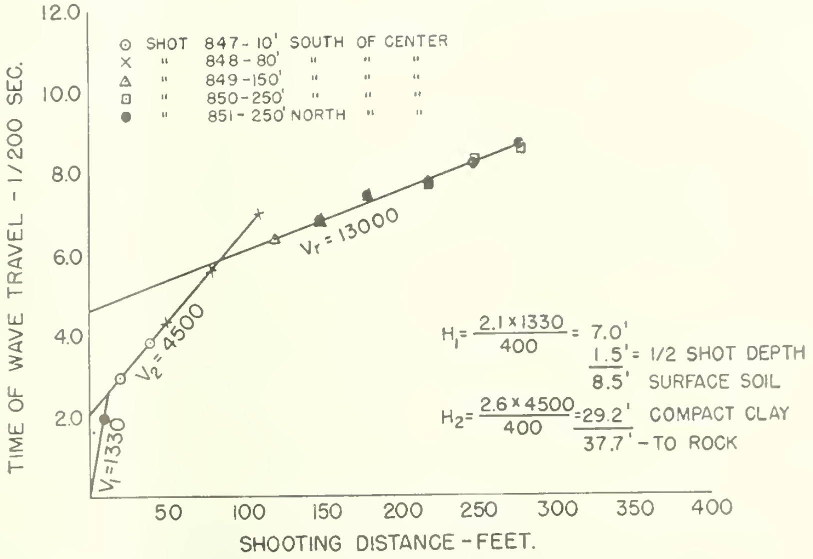


Figure 4.—Time-distance graph for seismic records shown in figure 1.

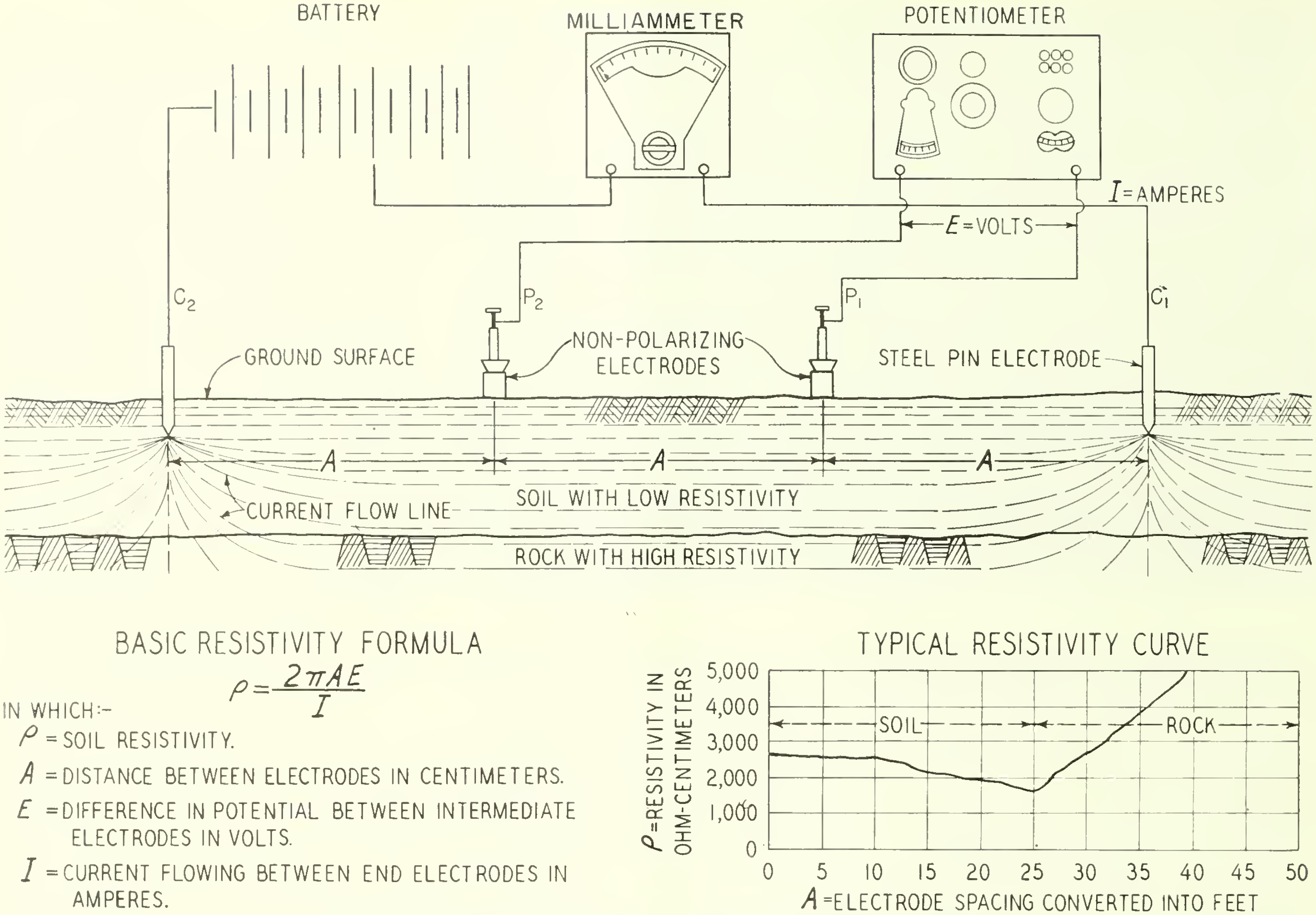

BATTERY
MILLIAMMETER
POTENTIOMETER
I = AMPERES
E = VOLTS
C2
P2
P1
C1
GROUND SURFACE
NON-POLARIZING ELECTRODES
STEEL PIN ELECTRODE
A
A
A
CURRENT FLOW LINE
SOIL WITH LOW RESISTIVITY
ROCK WITH HIGH RESISTIVITY
BASIC RESISTIVITY FORMULA
ρ = 2πAE / I
IN WHICH:-
ρ = SOIL RESISTIVITY.
A = DISTANCE BETWEEN ELECTRODES IN CENTIMETERS.
E = DIFFERENCE IN POTENTIAL BETWEEN INTERMEDIATE ELECTRODES IN VOLTS.
I = CURRENT FLOWING BETWEEN END ELECTRODES IN AMPERES.
TYPICAL RESISTIVITY CURVE
ρ = RESISTIVITY IN OHM-CENTIMETERS
5,000
4,000
3,000
2,000
1,000
0
SOIL
ROCK
0
5
10
15
20
25
30
35
40
45
50
A = ELECTRODE SPACING CONVERTED INTO FEET

Figure 5.—Fundamental principles of the earth-resistivity method.

film records, are usually used; and the denominator in the foregoing equations becomes 400 instead of 2.

When the geologic conditions existing at a particular test location actually approach those assumed in a theoretical analysis of the data obtained from refraction seismic tests, there is a remarkable similarity between the field curves obtained and the theoretical curve as it appears in figure 3. This is illustrated by the time-distance curve shown in figure 4 which was prepared from the field data shown in figure 1, supplemented by two additional shots placed at greater distances from the detectors. The data for this graph were obtained in New England where a relatively thin layer of loose soil was underlain by glacial till resting upon solid rock.

Theory of Earth-Resistivity Method

Experience has demonstrated that many of the materials making up the earth's crust can be identified, in some degree at least, by their reaction to the flow of a direct current of electricity. This is an action of electrolytic nature in which the moisture in the soils and rocks, together with the dissolved impurities, gives to the several materials characteristic resistances to a current flow.[5] These characteristic resistances or resistivities may be used for locating and, to some degree, identifying subsurface formations. Figure 5 illustrates diagramatically the earth-resistivity test and the Wenner electrode configuration (*1*) used by most investigators. In this test a prediction of the character of the subsurface materials is attempted by measurements indicating the magnitude of the resistance to direct current flow. Ordinary moist soils containing moderate amounts of clay or silt, with some electrolytic agent more or less active, have a comparatively low resistance. In contrast, sand, gravel, extremely dry, loose soils, and solid rock usually have relatively high resistivity values. These classifications are too general to be useful, however, and it is very necessary to calibrate the instrument with tests made over local materials which can be identified by exposed faces, test pits, core-drill records, or other means. Curves obtained later for unknown conditions may then be compared with those for known conditions, and a prediction can be made as to the materials lying below the surface.

Explanation of operation

Referring to figure 5, an electric current is passed through the ground from a direct current supply, usually one or more radio C-batteries, using the two outside electrodes C_1 and C_2. Measurement is then made of the drop in potential between two intermediate points P_1 and P_2, symmetrically spaced at the third points between the current electrodes. The current flow is determined with the milliammeter and the voltage or potential drop with the potentiometer, from which the resistivity of the material is computed by use of the formula

$$\rho = 2\pi A \frac{E}{I}$$

in which

A is the electrode spacing, in centimeters.
E is the drop in potential, in volts.
I is the current, in amperes, flowing in the circuit.

There is an empirical relation such that the "effective" current flows within a depth below the surface equal to A. That is to say, if A equals 10 feet, the resistivity obtained with the formula represents an average of all material existing within 10 feet of the surface. Thus, as the electrode spacings of the system are expanded, the current flow lines encounter the deeper portions of the underlying formations, such as the rock formation in figure 5. This material, having an appreciably higher resistivity than the overlying soil, affects the average resistivity values, the effect of the lower bed increasing progressively as the test is carried to greater depths.

---

[5] For a detailed description of the apparatus and a more comprehensive discussion of the earth-resistivity method of test see references (*1, 3, 7, 15, 25, 26*).

When using the empirical method of interpretation proposed by Gish and Rooney (*2*) the apparent resistivity ρ_a, obtained by inserting the measured values of A, E, and I from the field tests in the formula for resistivity as given above, is plotted as the ordinate against the electrode spacing A as the abscissa. The inflections in the resulting curve are interpreted as indicating changes in the materials underlying the surface. Where clay overlays rock a curve similar to that shown in the lower right-hand portion of figure 5 is usually obtained. The depth of the surface soil is taken as the value of A (electrode spacing) at which the upward inflection of the resistivity curve occurs. This empirical solution has been used in analyzing data from many tests in the past. Cases were found, however, where the plotted curve was smoothly rounded, with no inflection point, affording no criterion for predicting the depth of the surface material. Another empirical method of analysis has been proposed (*25*) for interpreting such curves, a brief summary of which follows.

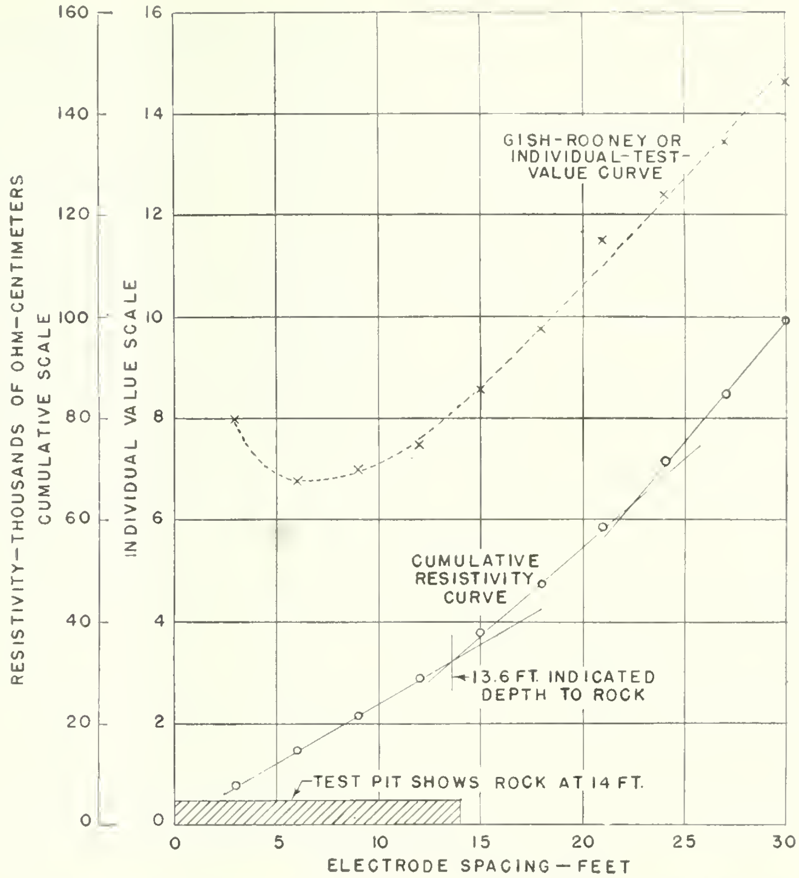


Figure 6.—Typical resistivity data, and analysis by use of the cumulative resistivity curve.

Empirical analysis

In figure 6 the smoothly rounded Gish-Rooney or individual-test-value curve is shown as a broken-line curve determined by the plotted crosses. The same field data are shown below this curve in the form of a cumulative resistivity curve determined by the plotted circles. When the values of apparent resistivity are plotted as a cumulative curve, a straight line or a curved line of gentle curvature is usually obtained so long as the "effective" current flow remains within the surface layer. When the electrode spacing is expanded to include increasing amounts of the deeper-lying rock formation the cumulative curve shows an increased curvature upward, reflecting the influence of the higher resistivity of the rock formation. It has been found that straight lines drawn through as many points as practicable on the cumulative curve, and intersecting in the region of increased curvature, will give a good approximation of the thickness of the surface material if the point of intersection of the straight lines is projected to the horizontal or dimensional axis. This is a purely empirical relation with no theoretical basis whatsoever. It has given rather close approximations of the depth of the surface layer in simple two-layer formations, however.

Referring to figure 6, it will be seen that the relatively shallow depth of 14 feet to rock, as determined by the test pit, affects strongly the measured values of apparent resistivity beyond an electrode spacing of about 10 feet. For this reason the plotted values of cumulative resistivity continue to show a rather marked degree of curvature well beyond what might be termed the critical point in the curve. The trend of the Gish-Rooney curve is used to determine the approximate critical point, which in this curve appears to be at an electrode spacing of 10–12 feet. Guided by the indications of the Gish-Rooney curve and such other correlating data as may be available from test pits or borings in the general area, the additional tangent intersections beyond the critical point may or may not be disregarded.

Other methods of analysis of earth-resistivity data based upon theoretical studies have been presented by Tagg (*7*), Hummel (*4*), Roman (*6*, *22*), Wetzel and McMurry (*20*), and others. Sets of theoretical curves for various assumed resistivities and thicknesses of the materials involved have been prepared for use by the operator as control in interpreting the field curves obtained. In some instances the field data are plotted to the same scale as that used in the theoretical curves, and on identical sheets, and are superimposed upon the theoretical curves. Where a fit is obtained by superimposition, the depths of the layers involved, as well as the resistivities of each layer, are obtained. Attempts to use these methods in analyzing the data obtained in the relatively shallow work done by the

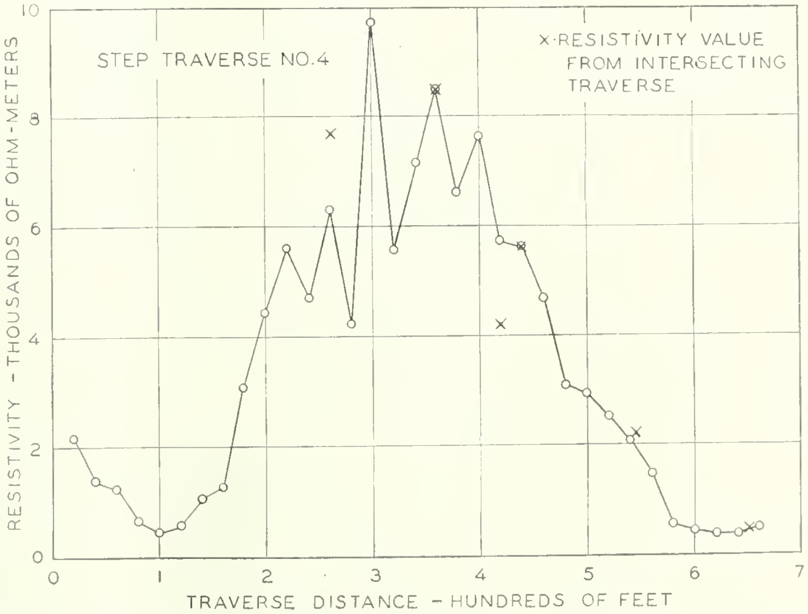


Figure 7.—Step traverse over a deposit of sandy gravel (electrode spacing, 20 feet).

Bureau of Public Roads have been discouraging, due to the time required for such studies and the frequency with which the field conditions failed to conform to those assumed in developing the theoretical curves. The empirical solutions heretofore described have been found to be more practical from the standpoint of time and cost in connection with a given exploration. This might be, in some cases, a deciding consideration between the geophysical tests and the direct methods of exploration ordinarily used.

Traverse surveys

When making surveys of areas, a somewhat different test procedure, one which might be termed the resistivity traverse or constant-depth traverse, is often used. In this, a succession of tests using a fixed electrode spacing is made along the selected traverse line, the interval between test sites being equal to the electrode spacing. The measured resistivity values are then plotted as ordinates against traverse distances as abscissas, and the resulting graph shows the variation in resistivity along the traverse line for a depth equal to the electrode spacing chosen. A typical example is shown in figure 7, the rise in resistivity between the 100- and 500-foot points on the traverse distance scale indicating the presence of higher resistance material within the depth explored. Traverse lines of this type, carried out systematically over an area, permit the preparation of a resistivity contour map such as that shown in figure 8. Such a map may be of considerable aid in rapidly locating and delineating critical areas that require more detailed study, or in locating valuable isolated deposits of granular materials or rock in areas where such materials are scarce.

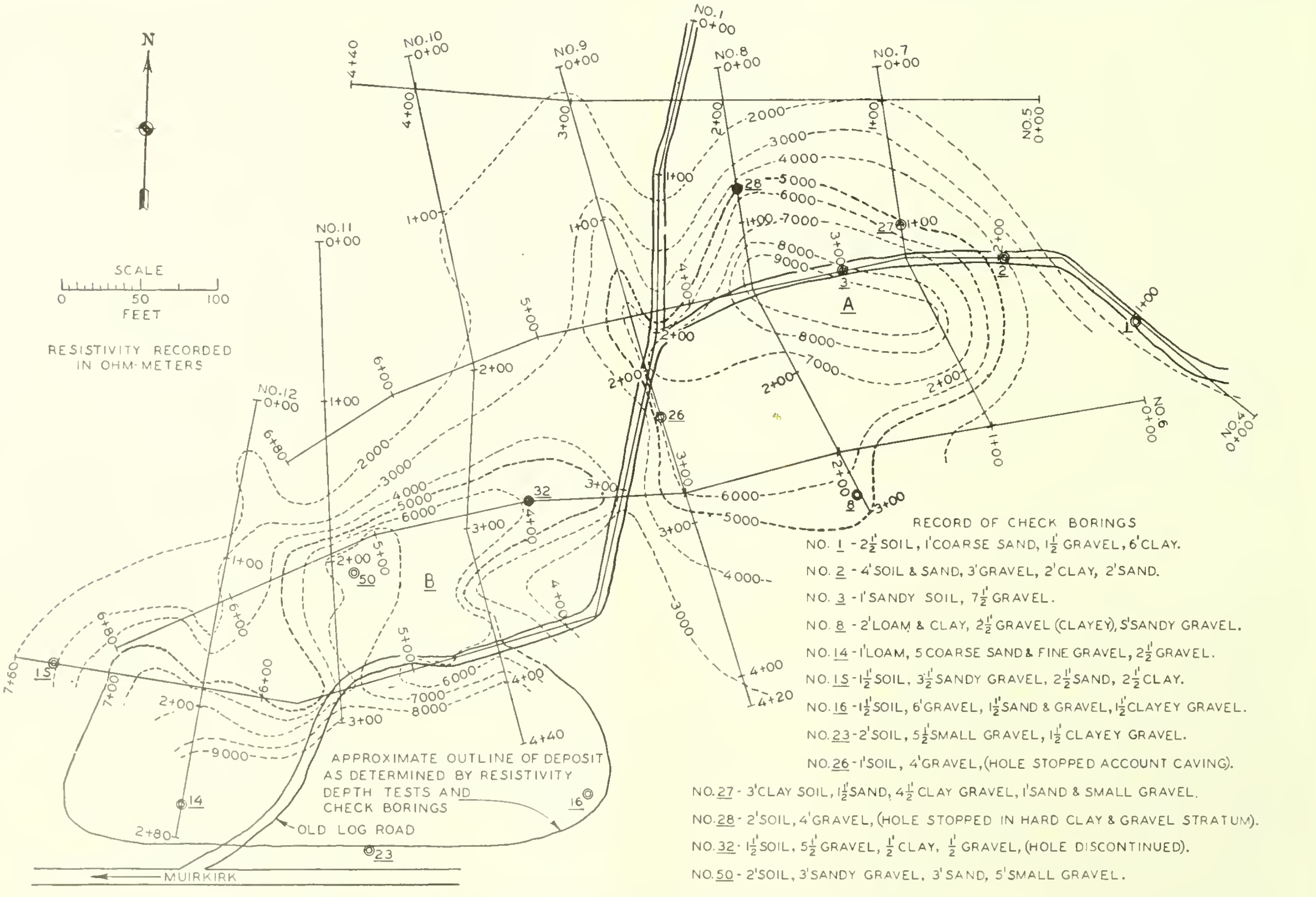
Rapid Subsurface Exploration Method Needed

Development during recent years of earth-moving equipment of ever-increasing capacity has made possible the quick and economical removal of huge quantities of excavation materials. Operating costs of such equipment are high, however, and a reasonably certain knowledge that the equipment selected will be able to handle all or a major portion of the materials on a given grading job, without costly delays from unforeseen adverse conditions, can be extremely helpful to contractors in establishing reasonable unit bid prices. A thorough investigation of the subsurface formation prior to design of slopes in cut sections will help to avoid the confusion that results when solid rock cuts, anticipated according to the plans, actually are found to be soil or other easily removable materials (or vice versa). Such errors in the classification of materials may lead to costly extra work or changes in design.

Stony soil, talus materials, and thin but continuous stringers of quartz or other hard materials extending throughout a cut may present insurmountable difficulties when attempting to explore subsurface conditions with hand- or power-operated auger equipment. Such troublesome conditions may result in misleading data when the auger is used but will not affect the data obtained with geophysical tests to any appreciable extent. For this reason, preliminary surveys by geophysical methods can be used to considerable advantage in determining the over-all character of the materials to be excavated. Complete and dependable information will make unnecessary hurried changes of alinement and grades to care for increased or decreased quantities of excavation materials, with possible delays of construction operations.

Application of Tests to Highway Problems

It has been found that both seismic and resistivity methods of test are practical for use in the study of many highway construction problems. The earth-resistivity apparatus, by reason of its simplicity of operation and the rapidity with which the shallow tests can be made, is believed to have a more universal application than does the seismograph. Accordingly, when making a detailed geophysical survey of a grading project, it has been the



RECORD OF CHECK BORINGS

NO. 1 - 2½' SOIL, 1' COARSE SAND, 1½' GRAVEL, 6' CLAY.
NO. 2 - 4' SOIL & SAND, 3' GRAVEL, 2' CLAY, 2' SAND.
NO. 3 - 1' SANDY SOIL, 7½' GRAVEL.
NO. 8 - 2' LOAM & CLAY, 2½' GRAVEL (CLAYEY), 5' SANDY GRAVEL.
NO. 14 - 1' LOAM, 5 COARSE SAND & FINE GRAVEL, 2½' GRAVEL.
NO. 15 - 1½' SOIL, 3½' SANDY GRAVEL, 2½' SAND, 2½' CLAY.
NO. 16 - 1½' SOIL, 6' GRAVEL, 1½' SAND & GRAVEL, 1½' CLAYEY GRAVEL.
NO. 23 - 2' SOIL, 5½' SMALL GRAVEL, 1½' CLAYEY GRAVEL.
NO. 26 - 1' SOIL, 4' GRAVEL, (HOLE STOPPED ACCOUNT CAVING).
NO. 27 - 3' CLAY SOIL, 1½' SAND, 4½' CLAY GRAVEL, 1' SAND & SMALL GRAVEL.
NO. 28 - 2' SOIL, 4' GRAVEL, (HOLE STOPPED IN HARD CLAY & GRAVEL STRATUM).
NO. 32 - 1½' SOIL, 5½' GRAVEL, ½' CLAY, ½' GRAVEL, (HOLE DISCONTINUED).
NO. 50 - 2' SOIL, 3' SANDY GRAVEL, 3' SAND, 5' SMALL GRAVEL.

Figure 8.—Resistivity contour map over a deposit of sandy gravel.

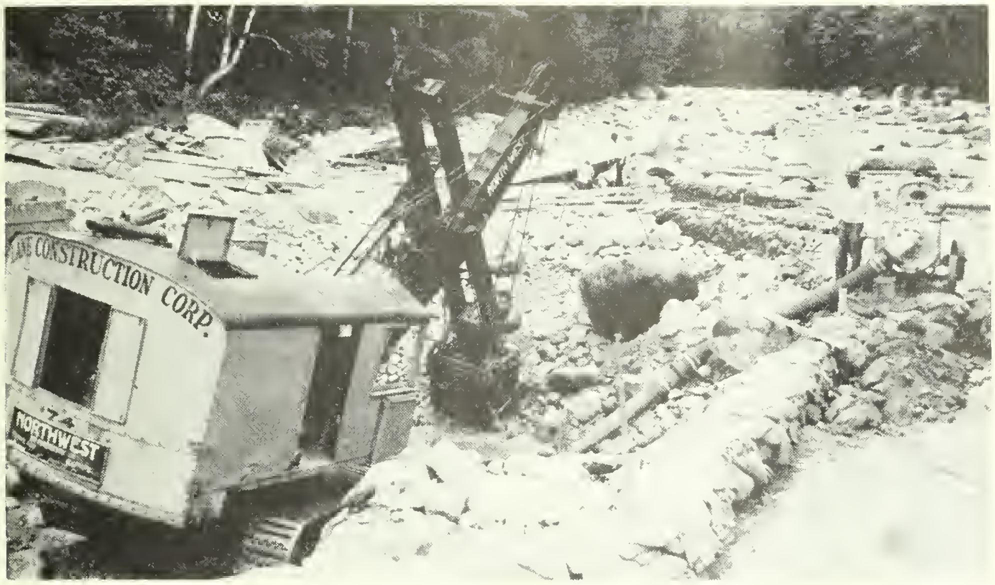


Figure 9.—Tightly cemented boulder formation predicted by seismic tests at Pemigewasset River crossing near Lincoln, N. H.

practice of the Bureau of Public Roads to make a resistivity survey first and, if necessary, to follow with a limited number of check tests with the seismograph in areas where the resistivity data fail to identify the subsurface formations adequately. This procedure has proved to be very satisfactory in field investigations of 10 construction projects ranging from 1½ to 12 miles in length, located in Arkansas, Georgia, Missouri, North Carolina, Tennessee, Virginia, and the District of Columbia. Reports have been received on four of these projects which have since been constructed, and the conditions found during construction were substantially as predicted from results of the geophysical tests.

Results of Seismic Tests

In seismic tests, the velocity of the transmitted sound waves generally increases with an increase in the density of the transmission medium. Wave velocities in loose, unconsolidated soil layers range from 600 to 1,500 feet per second. Velocities in more compact subsurface layers range from 2,000 to 9,000 feet per second, the lower ranges of 2,000 to 3,500 usually being associated with clay materials and the higher ranges of 4,000 to 9,000 with compact gravels, badly broken or weathered rock, and soil-boulder mixtures. Solid rock usually allows wave transmission velocities between 10,000 and 20,000 feet per second, depending upon the type of rock and its degree of weathering or fracture. In predicting the character of material that may be found, particularly in the intermediate velocity group (4,000 to 9,000 feet per second), considerable judgment, as well as some knowledge of local geologic conditions, is required. Calibration tests over known subsurface formations are essential for a successful interpretation of the data obtained.

Actual identification of the materials involved is not always necessary, however. For example, broken rock or badly seamed rock, highly compacted shale, or cemented gravel, having similar velocity characteristics, may be expected to offer somewhat similar difficulties in excavation operations, possibly requiring some blasting and special handling and distribution. These same materials will probably show similar load-carrying capacities when considered for foundation purposes, particularly where surrounded by materials which have been left in an undisturbed state.

Use at bridge sites

As an example, seismic tests made in New Hampshire at a proposed bridge site on the Pemigewasset River, near Lincoln, showed a comparatively high wave velocity for material lying only a few feet below the surface and apparently continuing to a depth of at least 40 feet. This material, with a wave velocity of 9,400 to 9,600 feet per second, was predicted to be a tightly cemented boulder formation with excellent load-carrying capacity. Figure 9 shows the excavation subsequently made for one of the bridge piers at this location. The material was so tightly cemented together that only a simple sandbag cofferdam was required Soundings and drill holes through material of this type would be impossible or could be made only with great difficulty and at considerable cost.

Another bridge location, near Crater Lake in Oregon, was investigated by the seismic method in about 3 hours. The data obtained showed the subsurface formation to be a very dense material providing a wave velocity of 8,400 to 8,600 feet per second. Here, again, there could be no doubt regarding the existence of adequate foundation materials. Figure 10 shows the seismic data for two of the three tests made at this location.

Data for slope design

Experience is needed to determine the particular slope design that will be adequate where certain materials within a local area are involved. With proper calibration data, the seismic method often can be relied upon to establish definitely the presence of these materials. As an example, the data in figure 11 show the presence of and depth to the predominant material, shale.

As mentioned previously, portability of equipment is of primary importance to the successful application of geophysical methods of test in preliminary surveys for a highway location. Figure 12 shows typical terrain encountered in the construction of roads in National parks and forests in various parts of the country. In designing a modern highway through such country, any information regarding the materials likely to be encountered in excavating cut sections is important. A close

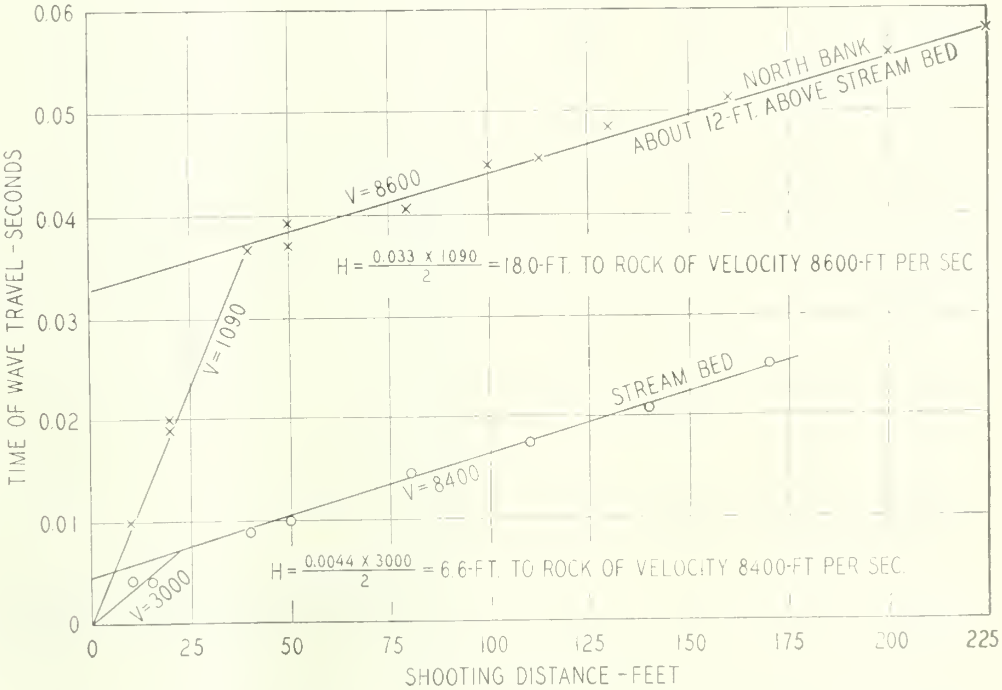


Figure 10.—Time-distance graph from two seismic tests at a bridge site near Crater Lake, Oreg.

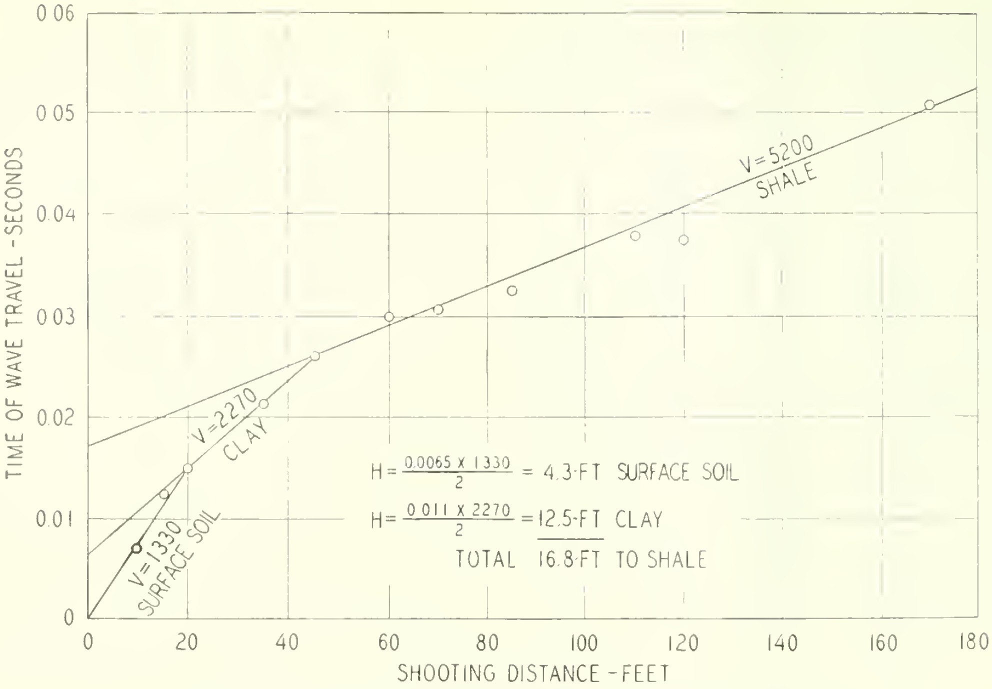


Figure 11.—Refraction seismic test over a shale formation.

balance of quantities must be maintained both in the interests of economy and to avoid waste or borrow areas which would mar the natural scenic beauty of the roadside. Therefore, a design prepared for solid rock, with a ½ to 1 slope in cut section (such as the one shown in figure 12), could lead to embarrassing difficulties should a comparatively loose earth or talus material be encountered, requiring a 1½ to 1 slope reaching high up the mountainside. Large quantities of material would have to be wasted or cared for by substantial changes in alinement and grade. Conversely, where earth slopes are expected and rock is found, a source of borrow would be required for adjacent large fills unless major grade changes were made.

The ridge from which the photograph shown in figure 12 was taken originally had been assumed to contain solid rock. A tunnel several hundred feet in length was proposed to carry the roadway through the ridge, some 100 feet below the top. Test pits dug to obtain design data for portal construction failed to encounter rock above grade. Several weeks were required for this exploration work, which cost hundreds of dollars, and finally a redesign for an open cut was found necessary. Seismic tests requiring no more than 2 or 3 hours were sufficient to establish the fact that no solid rock existed in the hill. The excavation during construction was made with the usual heavy earth-moving equipment. Studies made with seismic equipment at other sites have been of value in portal design and in indicating the probable need for tunnel lining.

Slide conditions

Another problem to which refraction seismic equipment has been applied occurs in regions where slide conditions are prevalent. In some cases the loose talus material frequently involved in a slide rests upon a sloping shale formation which constitutes the sliding surface. This talus material has velocity characteristics differing from those of the more compact shales, making possible the location of the plane of separation.

Although the refraction seismic test has proved of value in preliminary surveys in various phases of highway construction, as has been pointed out, it has not been used to the same extent as the earth-resistivity test in recent years because of the greater time required for a seismic test. Six or eight seismic tests per 8-hour day is about the maximum number to be expected, under reasonable field conditions. Fifteen to twenty resistivity tests are usually possible under similar field conditions. Seismic tests can be utilized as a completely independent check of the indications of the more rapid resistivity tests, however, and are used for this important purpose in the routine work done by the Bureau of Public Roads.

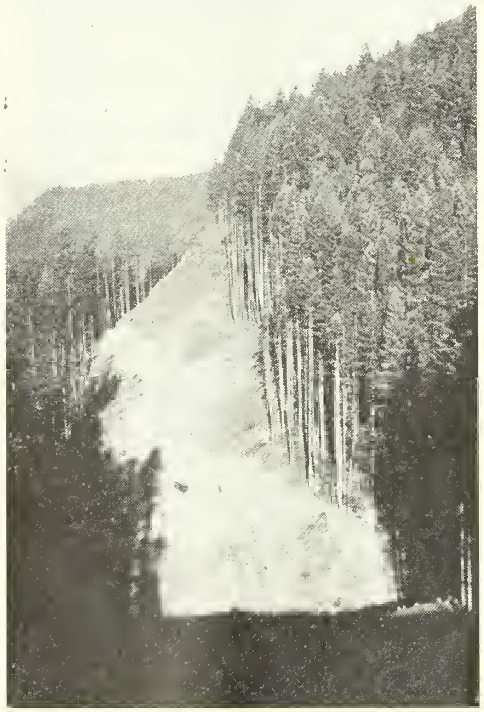


Figure 12.—Rugged terrain in a National forest where portable equipment for subsurface exploration is invaluable.

Results of Earth-Resistivity Tests

In a subsurface survey in the field, it is an established procedure to make calibration tests with the resistivity apparatus over exposures of formations believed to be typical of those in the area of immediate interest. Resistivity curves for the known conditions are then used for comparison with curves obtained over unknown conditions elsewhere in the area. From these comparisons reasonably accurate predictions can be made regarding the materials to be encountered below the surface, and their location. Figure 13 shows typical resistivity curves obtained in Arkansas, in the Ozark National Forest, in the course of a resistivity survey of about 22 miles of proposed roadway. The calibration curves on the left were obtained for heavy sandstone ledges interbedded with shales and for the soils and decomposed shales typical of the region. These latter are materials that could be handled with the heavy self-loading scraper. The curves of the right-hand graph are examples of the field curves obtained in the survey along the right-of-way of the proposed roadway. Little difficulty was experienced in predicting the types of materials involved for the several curves shown. Figure 14 shows the two general types of material over which calibration tests were made.

Based upon the usual methods of direct exploration, the original slope design called for rock slopes over a considerable portion of the right-of-way. Actually, earth materials, as predicted from the results of the resistivity survey, were found in a majority of the cuts during the construction of about 14 miles of roadway thus far completed. The entire 22 miles was investigated in about 12 working days, one 8-mile section being covered in 3½ days.

In northwest Georgia, resistivity calibration tests over solid rock and over earth formations produced curves as shown in the left and in the lower right-hand graphs, respectively, of figure 15. Although the shapes of the curves obtained are quite different from those obtained for materials of the same general classification in Arkansas, the two materials, rock and earth, can very easily be distinguished one from the other. On the basis of these calibration data, the typical field curves shown in the upper right-hand corner were all interpreted as identifying earth materials easily removed by self-loading scrapers. Figure 16 shows the two types of material over which calibrations were made.

In the Great Smoky Mountains National Park in western North Carolina, the dense granite rock formations typical of that area weather into a highly micaceous decomposed rock material that can be removed with scraper units. As shown by the calibration curves in figure 17 (solid-line curves), this material has an extremely high resistivity, 1.5 million ohm-centimeters, which is ten times as great as resistivities found in some solid rock in other parts of the country. Due to the fact that the parent rock in a solid, unweathered state has even higher resistivities (4 to 5 million ohm-centimeters), it is again possible to differentiate between earth and rock excavation. The appearance of the materials over which the calibrations were obtained is shown in figure 18. That section of the Blue Ridge Parkway on which the resistivity survey was made has not yet been built and no confirming correlations are available at the present time.

In southeast Missouri, the porphyry rock found in the vicinity of Farmington has a resistivity as indicated by the upper curve of figure 19, while a calibration test over the soil common in the same area produced the lower curve of the figure, indicating almost no resistance to direct current flow. No difficulty was encountered in determining the type of material present in all but one cut of all those investigated on a 4-mile section.

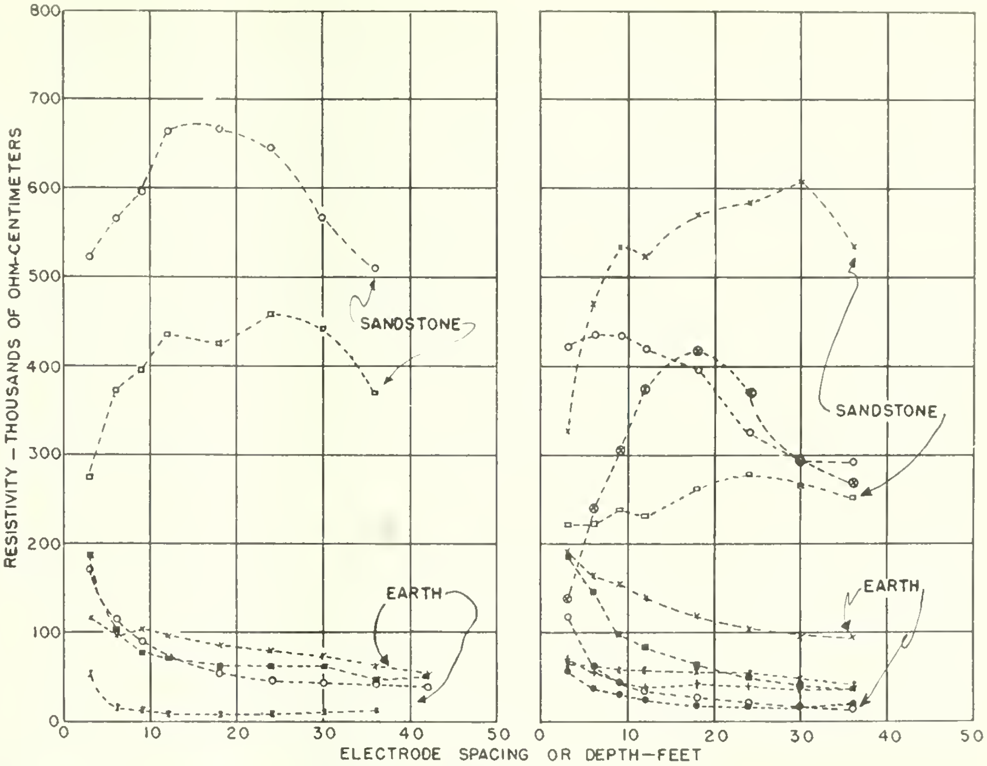


Figure 13.—Resistivity calibration curves (left) and typical field curves (right) obtained in the Ozark National Forest in Arkansas.

Other resistivity surveys on construction projects in Maryland, Tennessee, Virginia, and the District of Columbia provided information regarding the subsurface formations that agreed closely with conditions actually found during construction.

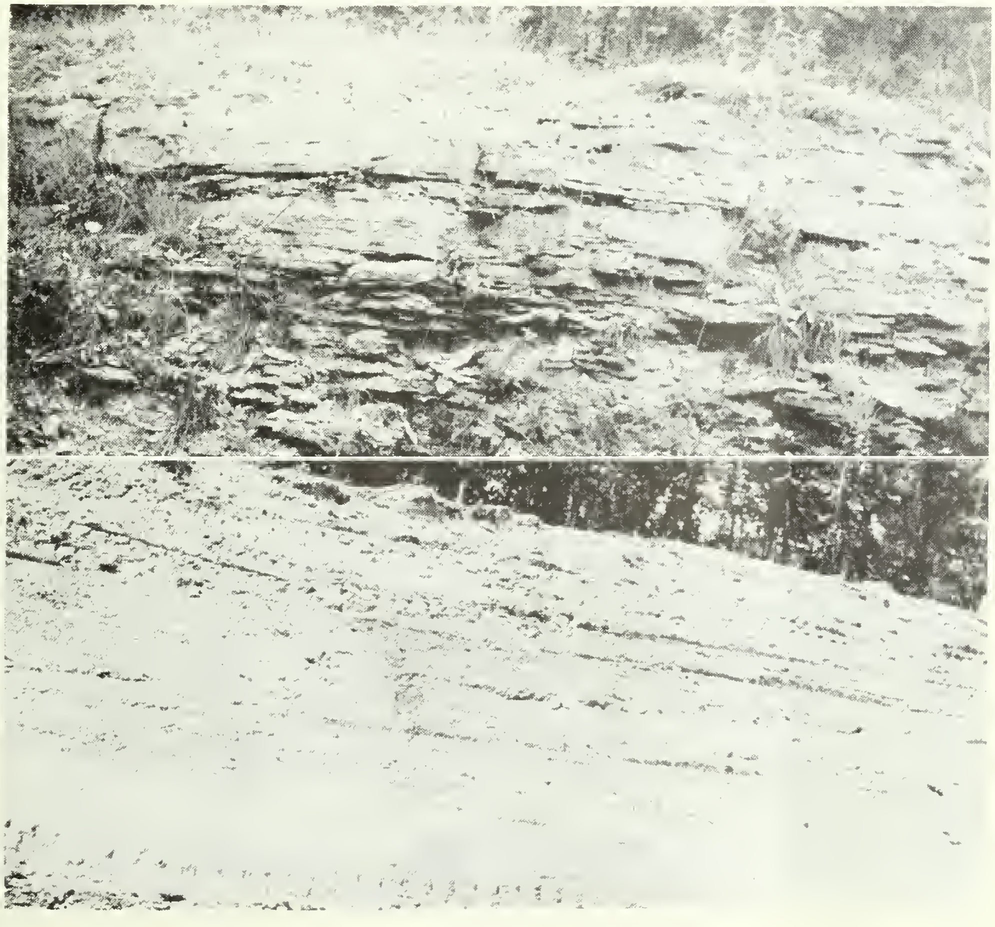


Figure 14.—Locations where resistivity calibration curves shown in figure 13 were obtained over rock (upper) and earth (lower).

Application to foundation problems

Earth-resistivity tests can be of assistance also in a subsurface study of the foundation conditions existing at proposed building sites, bridge locations, and in other areas where solid rock foundations are required or desirable.

In 1942, at the request of the Navy Department, a resistivity survey was made of a 150-acre tract at Carderock, Maryland. The site is underlain with rock and information was desired as to the depth to rock throughout the reservation. Altogether, over 500 depth tests and upwards of 10½ miles of constant-depth resistivity traverse were made in carrying out the survey. From the information obtained a rock contour map (fig. 20) was drawn up showing probable rock elevations on 2-foot contours over the entire area. An accuracy of ±2 feet at any point in the area mapped was predicted. In 1944 an existing building with a width of 120 feet was extended for 1,800 feet in the area that had been mapped. Cross sections of the rock surface as found, obtained at intervals of 10 feet along the building axis, showed a difference in total amount of stripping of less than 6 percent from that computed from the rock contour map prepared in 1942. About 100,000 cubic yards of stripping were involved.

Figure 21, showing typical traverse data obtained in this study, illustrates how the resistivity test can be used in a preliminary survey to obtain information that may be used to guide a detailed survey by borings and eliminate many unnecessary soundings or borings. The flat-lying portion of the curve suggests a uniform condition for much of the distance traversed. The peaks in resistivity indicate those areas where direct borings

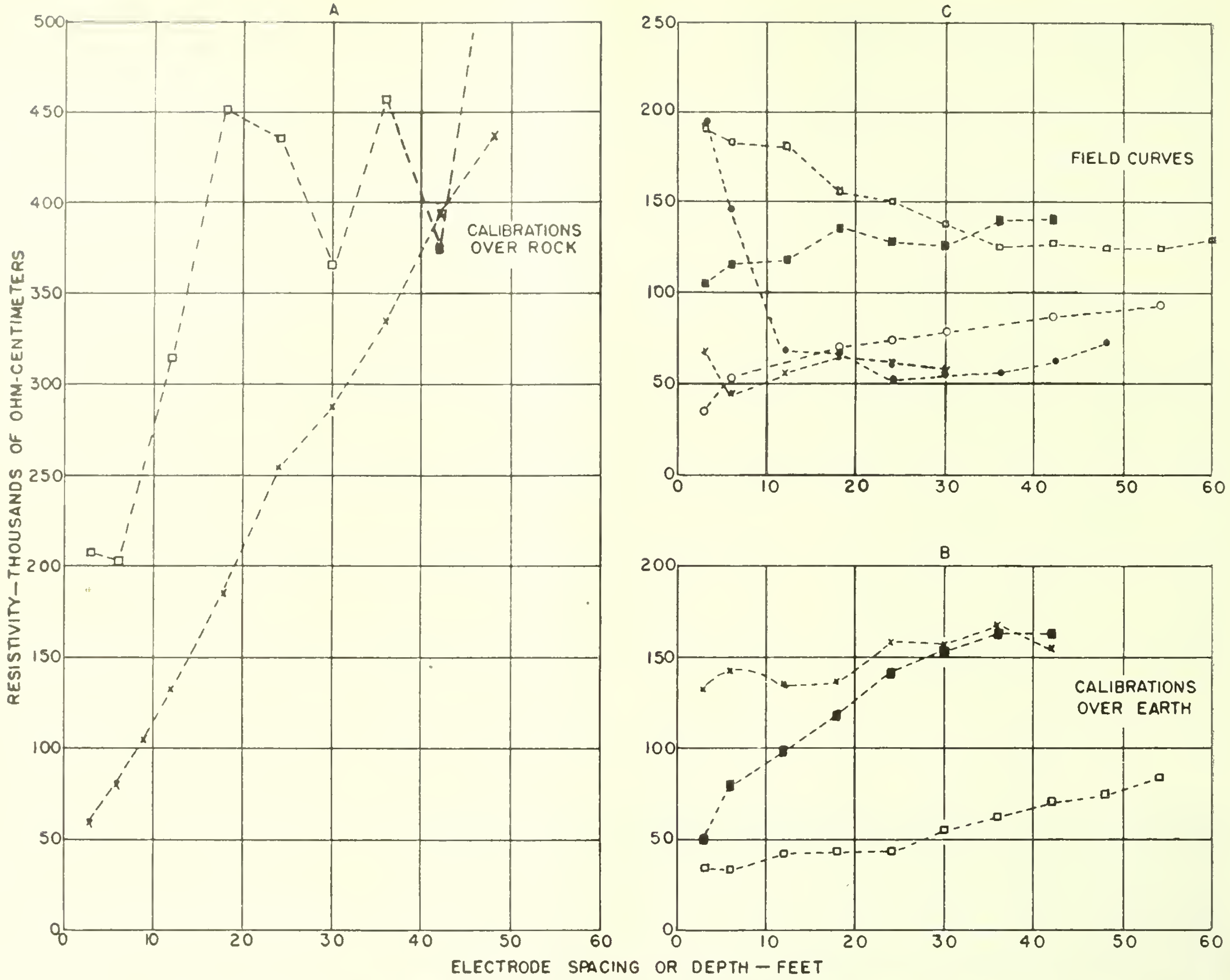


Figure 15.—Resistivity calibration curves and typical field curves obtained in northwest Georgia.

should be concentrated to delineate in detail the obvious anomaly. These buried ridges of rock can be traced across wide areas, indicating regions where excavation will be difficult or where foundation conditions will be excellent at shallow depth. The underlined dimension figures shown are depths to solid rock obtained by resistivity depth tests made at 100-foot intervals along the line of the traverse. The two depth curves shown in the inset are a striking indication of radical changes in the subsurface at stations 2+00 and 13+00 of the traverse.

In bridge foundation studies there have been numerous instances when the routine subsurface survey, using the usual methods of probing, wash boring, or drilling, has failed to disclose unusual conditions later found during construction. Piers designed originally for solid rock foundations have had to be carried to considerably greater depths than those shown on the original plans, or supported upon piling extending to rock at a lower elevation. Figure 22 shows several resistivity depth curves obtained in a postconstruction survey of a bridge crossing of the Flint

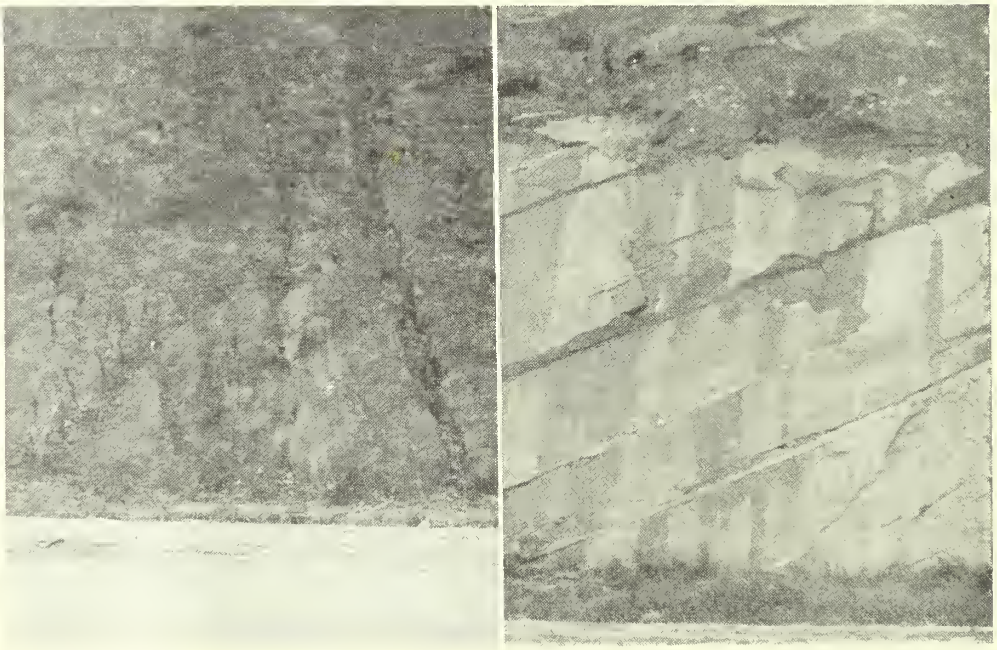


Figure 16.—Locations where resistivity calibration curves shown in figure 15 were obtained over earth (left) and rock (right).

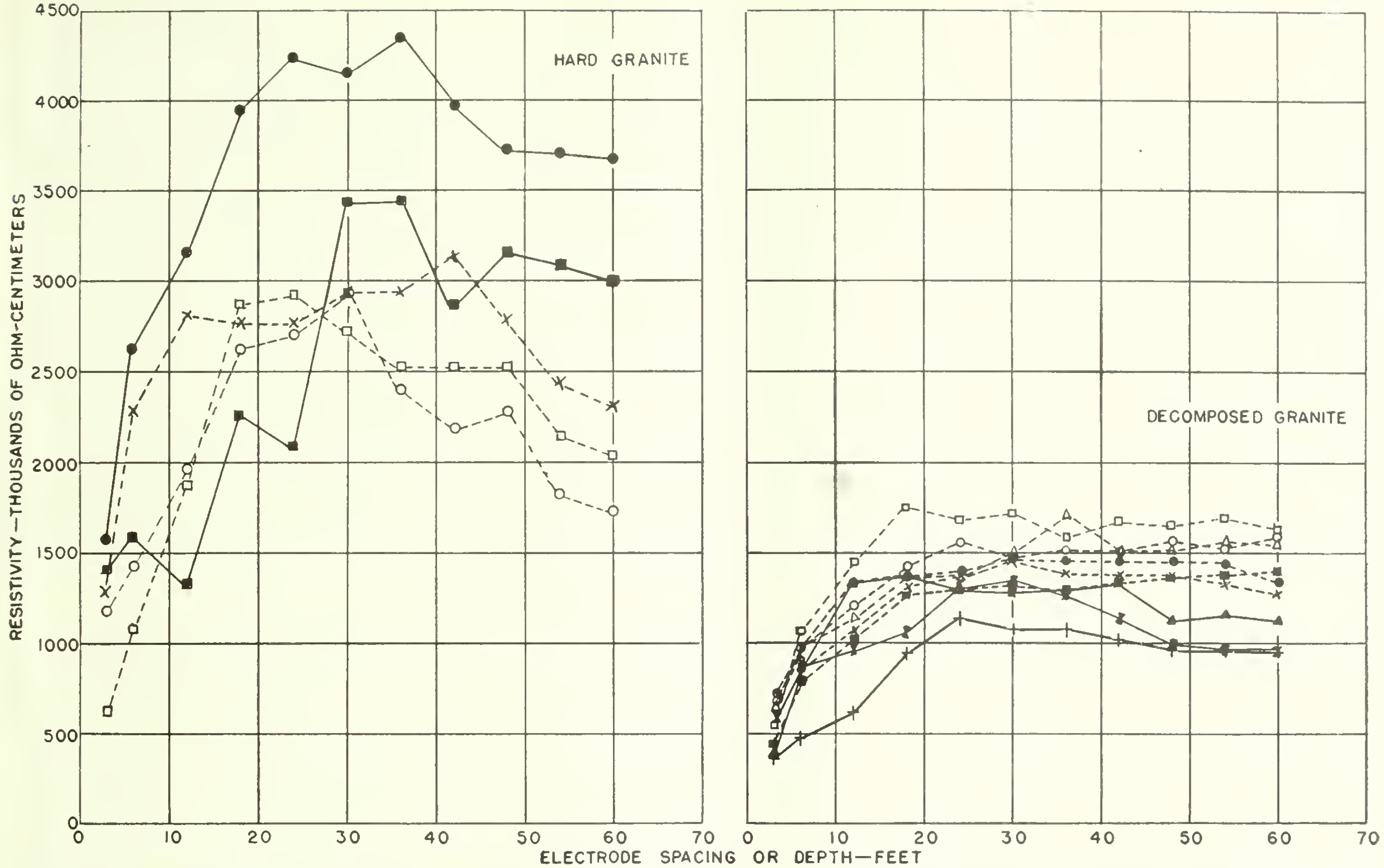


Figure 17.—Resistivity calibration curves (solid lines) and typical field curves (broken lines) obtained over solid and decomposed granite in North Carolina.

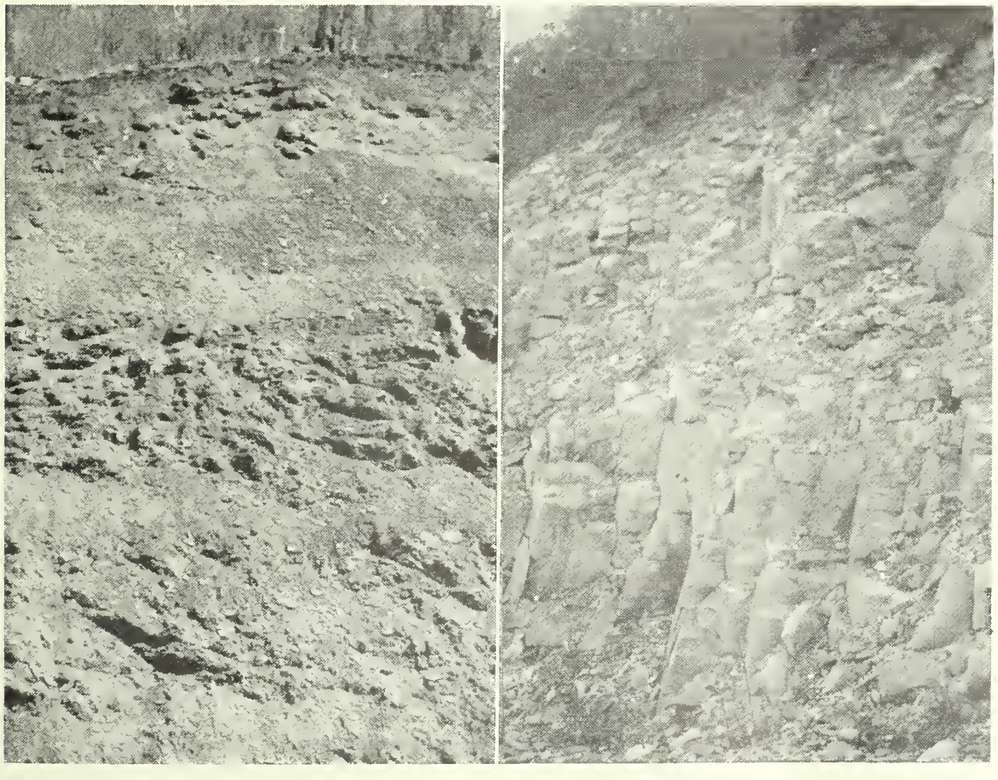


Figure 18.—Locations where resistivity calibration curves shown in figure 17 were obtained over decomposed granite (left) and solid granite (right).

River in southwest Georgia. The individual graphs show the plan data for depth to rock, the depth to rock as found during construction, and the depth to rock as predicted from the resistivity data. The general agreement between the results of the resistivity tests and the actual conditions existing is apparent.

Although it is not possible to make an unqualified statement regarding the effectiveness of the resistivity test generally in all localities and under all possible combinations of geologic formations, the fact remains that 1 or 2 hours' work at a particular location will usually determine the extent of its usefulness in solving the particular problem at hand. The data from the tests made in Georgia are similar to those that have been obtained elsewhere in areas where the river deposits have shown resistivity characteristics differing from that of an underlying rock formation.

Tests in swampy areas

The investigation of swamps, peat bogs, and salt-marsh areas by geophysical tests probably constitutes a marginal application of such methods, since simple probings are often effective in these areas. However, since a resistivity depth test to depths of 60 feet can be made in a period of 12 to 15 minutes, the deeper muck deposits can be studied economically in competition with direct probings. Where sand lenses are likely to be present

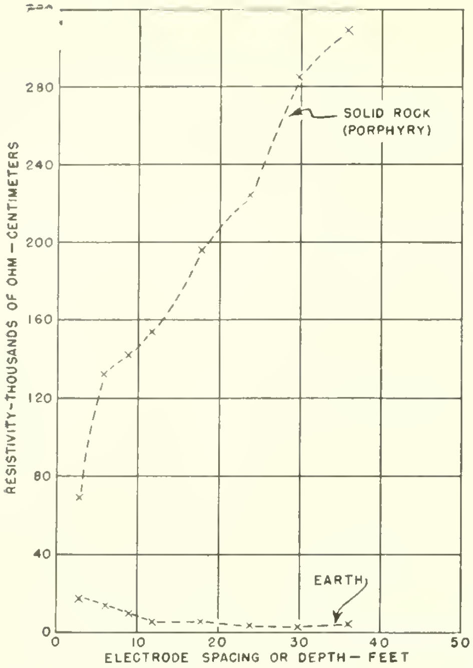


Figure 19.—Typical resistivity calibration curves over solid rock and earth formations in southeastern Missouri.

within a relatively deep layer of muck or peat, probings can result in erroneous information, being stopped by relatively thin sand layers. The resistivity test, due to the large volume of material involved, will not be appreciably affected by thin sand lenses and will indicate depth to a true bottom formation.

The curves shown in figure 23 were obtained in a study of the application of resistivity tests to determine the depth of peat bogs. This study, carried out in Michigan in 1941, confirmed earlier test results obtained in a study of peat formations in Wisconsin as reported by Kurtenacker (*9*, *10*) and demonstrated that resistivity tests can be used successfully in determining the depth of peat and muck layers.

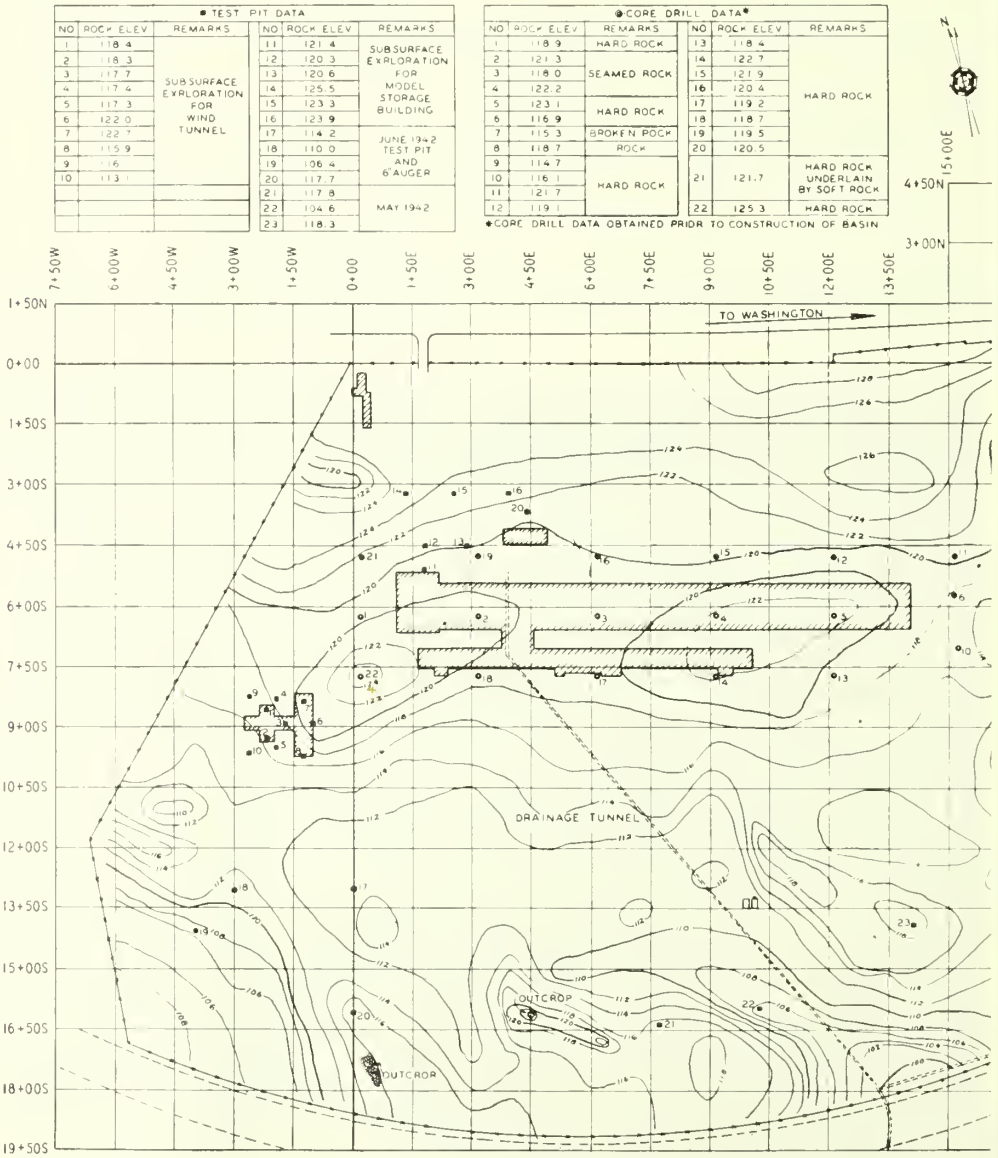
Figure 24 shows results of a resistivity survey along a taxiway at the Washington National Airport. The resistivity tests not only indicated the bottom of the floating sand-gravel fill upon which the taxiway was placed, but they also rather effectively located a second horizon comprising the sand and gravel bed of the river. The conditions as they exist were determined by data obtained from the auger holes shown in the figure. The hatched portions of the columns representing the bore holes denote the thickness of the sand-gravel fill, and the solid black portions indicate the thickness of the relatively soft silt on the river bottom.

It is of interest to note the resistivity peaks occurring in the 10-foot depth resistivity traverse, shown in the lower portion of the figure, which coincide with the thicker portions of the granular fill. Even the small difference of a few feet in the over-all depth of the muck from place to place had caused differential settlement sufficient to affect the pavement of the taxiway.

Knowledge of local geology essential

Just as with the seismic test, a working knowledge of the local geology is necessary when attempting to predict the actual character of the materials below the surface from resistivity tests. Figure 25 shows two resistivity traverses, the upper one made over a rock ridge rising almost to the surface, the lower one made over a sand and gravel deposit. The similarity of the two curves might lead to error in predicting the type of material without at least a general knowledge of the local geology. The depth tests shown on the right of figure 25, however, offer some clue as to the actual material involved. When a solid rock formation is present beneath a soil overburden a sharply rising curve is usually obtained, similar to the curve shown in the upper right-hand graph. The dipping curve shown in the lower right-hand graph for the sand and gravel formation suggests to the experienced operator, acquainted with the local geology, that such a formation is likely to be present. Descending curves of this same general type obtained in Arkansas, however, might involve a sandstone ledge underlain by decomposed shales. Or, in southwest Colorado, a curve of this type might be obtained with talus material overlying low-resistivity shales. It is necessary to depend upon a study of local geology and upon calibration tests over exposed materials in the same region when attempting a classification of the materials involved.

In Pennsylvania, a depth test was made at the location of a proposed drill hole in an investigation of foundation conditions for a bridge to carry the Pennsylvania Turnpike across the Susquehanna River. The resistivity depth-test data indicated a definite change at a depth of 27 feet, as shown in figure 26. The consultant geologist suggested that the underlying formation might be shale. The data for this curve were obtained in about an hour's time. In contrast, a drill crew, starting simultaneously with the resistivity test, spent 2½ days in reaching the shale at 26.5 feet.



● TEST PIT DATA

NO	ROCK ELEV	REMARKS	NO	ROCK ELEV	REMARKS
1	118.4	SUBSURFACE EXRLORATION FOR WIND TUNNEL	11	121.4	SUBSURFACE EXRLORATION FOR MODEL STORAGE BUILDING
2	118.3		12	120.3	
3	117.7		13	120.6	
4	117.4		14	125.5	
5	117.3		15	123.3	
6	122.0		16	123.9	
7	122.7		17	114.2	JUNE 1942 TEST PIT AND 6" AUGER
8	115.9		18	110.0	
9	116		19	106.4	
10	113.1		20	117.7	
			21	117.8	MAY 1942
			22	104.6	
			23	118.3	

◉ CORE DRILL DATA*

NO	ROCK ELEV	REMARKS	NO	ROCK ELEV	REMARKS
1	118.9	HARD ROCK	13	118.4	HARD ROCK
2	121.3	SEAMED ROCK	14	122.7	
3	118.0		15	121.9	
4	122.2		16	120.4	
5	123.1	HARD ROCK	17	119.2	
6	116.9		18	118.7	
7	115.3	BROKEN ROCK	19	119.5	
8	118.7	ROCK	20	120.5	
9	114.7	HARD ROCK	21	121.7	HARD ROCK UNDERLAIN BY SOFT ROCK
10	116.1				
11	121.7				
12	119.1		22	125.3	HARD ROCK

*CORE DRILL DATA OBTAINED PRIOR TO CONSTRUCTION OF BASIN

Figure 20.—Part of a rock contour map prepared from data obtained by earth-resistivity tests.

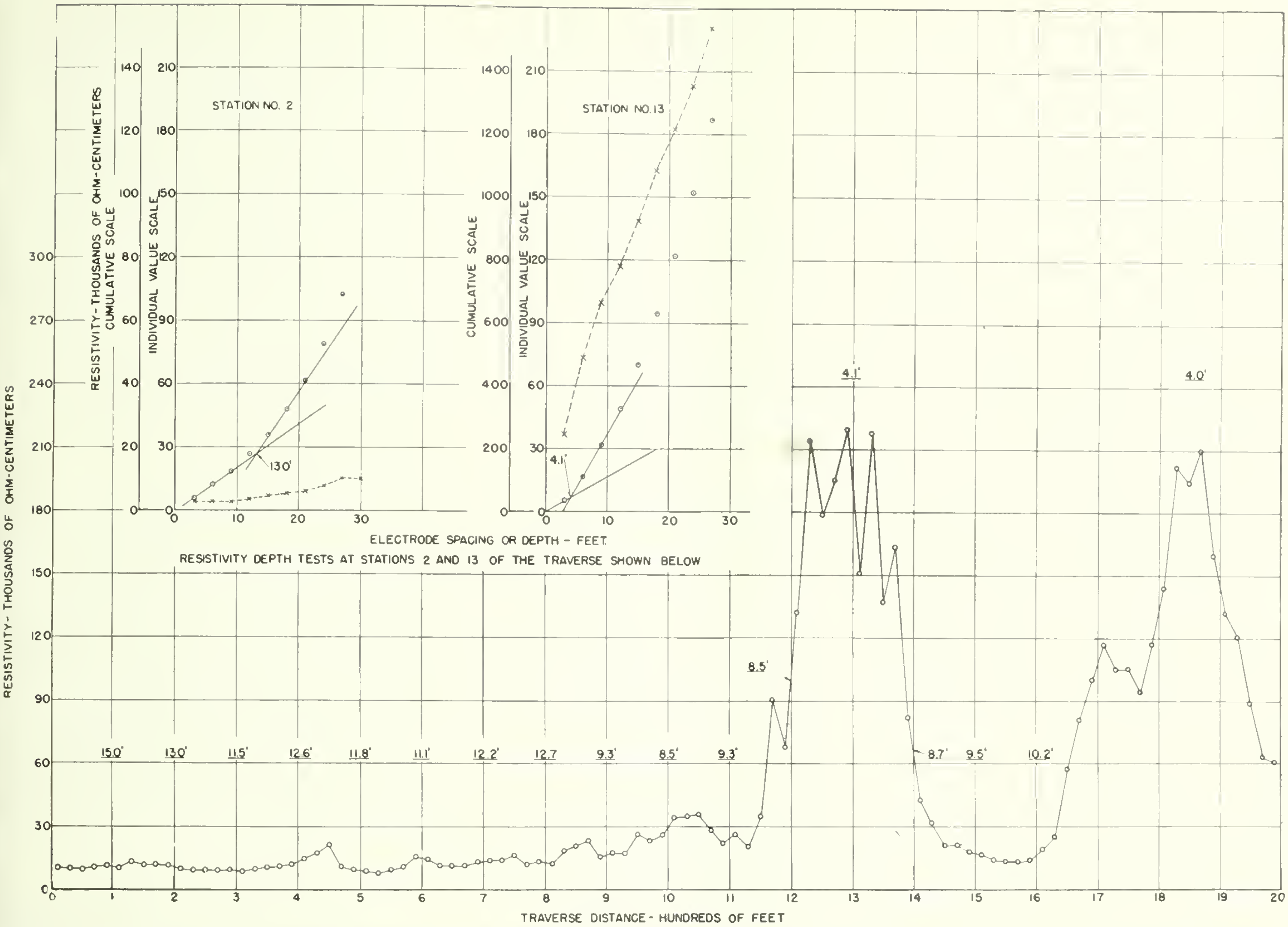


Figure 21.—This earth-resistivity constant-depth traverse discloses abrupt changes in rock surface underlying a clay soil overburden. The traverse involved a 20-foot depth along a 2,000-foot line. The underlined figures show results of resistivity depth tests for depth of overburden; curves for two such tests are shown in inset.

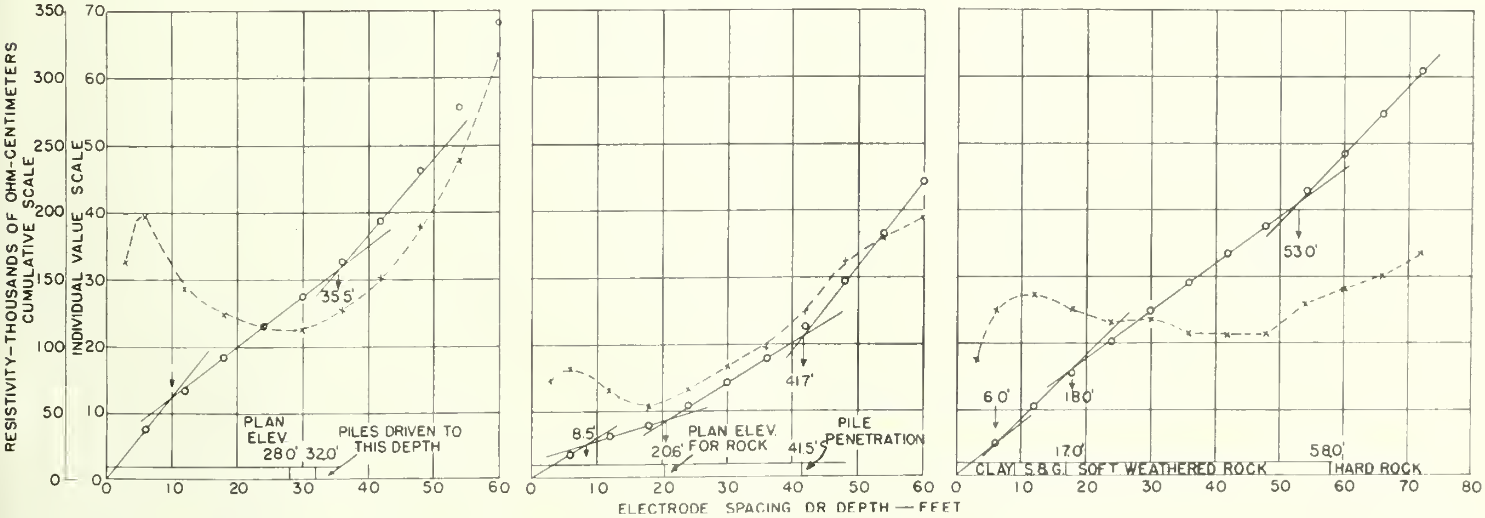


Figure 22.—These earth-resistivity tests at Flint River crossing in Georgia located the subsurface rock for bridge foundations more accurately than drilling.

Advantages and Limitations of the Geophysical Methods of Test

The seismic test is particularly useful for determining the presence or absence of dense, solid rock. The high velocity usually associated with such formations makes the determination quite dependable. Although the resistivity test will, in most instances, indicate the depth of overburden to a high-resistivity formation such as rock, it cannot, in the absence of confirming geologic data, furnish a completely dependable basis for predicting the presence of rock in all cases. As has been shown, sand and gravel under special conditions can have reasonably high resistance and show subsurface anomalies quite similar to those shown by solid rock. However, in areas where solid rock layers are interbedded with less dense materials such as shales, as occurs in Arkansas and in many other areas, the resistivity test is much the better tool since it is possible to detect the change from hard rock to softer, less resistant shales. The seismic test under such geologic conditions would be limited to an indication of the depth of overburden to the high-velocity sandstone or limestone and the lower-velocity shales could not be located. This results from the fact that the first wave to reach the detector will usually cause such high degree of activity in the galvanometer elements controlling the deflections of the photographed light rays as to preclude the possibility of detecting any subsequent wave arrival through the underlying low-velocity formation.

The resistivity test, particularly the resistivity traverse, offers a practical means for the rapid investigation of large areas in search of localized deposits of gravel, sand, or other granular materials useful in road construction. The method can be used also to determine the extent of special soils, such as impervious silty and clayey soils, which might be useful for earth-dam and levee construction.

The seismic test is not well adapted for an area survey, but is best applied to the determination of conditions at a single designated spot or limited area such as a dam site or bridge location. Even in the limited areas, if differential weathering has been in progress,

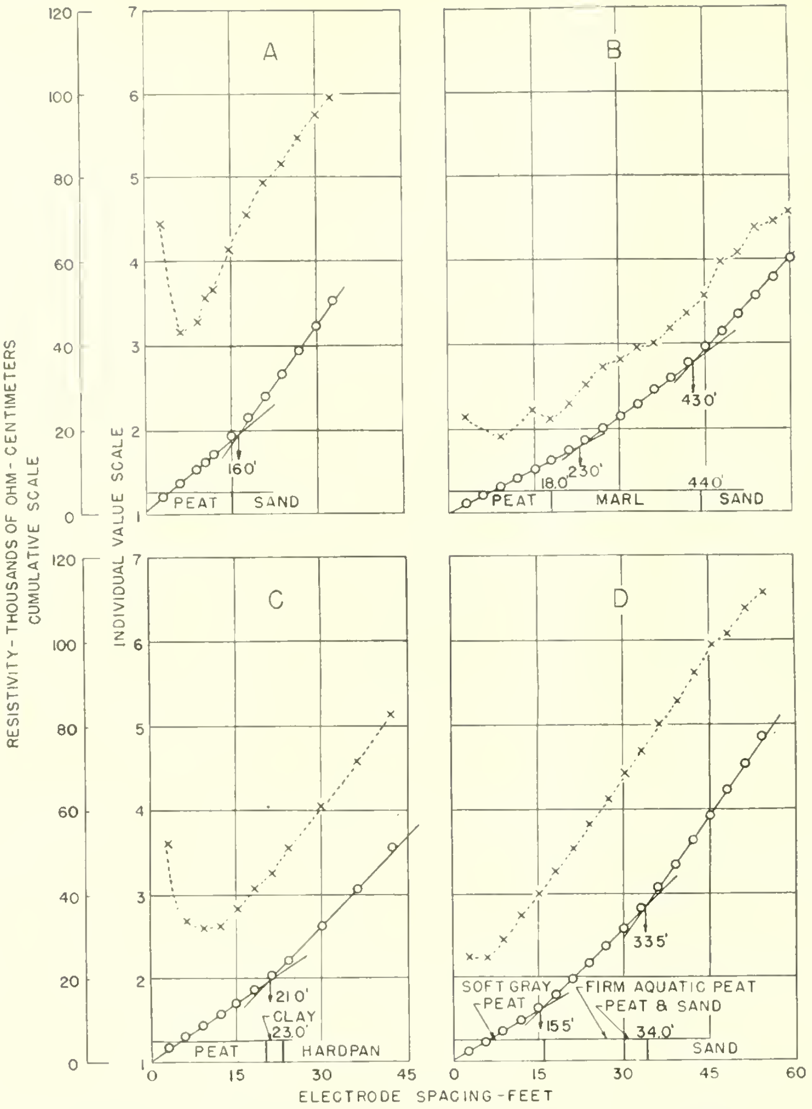
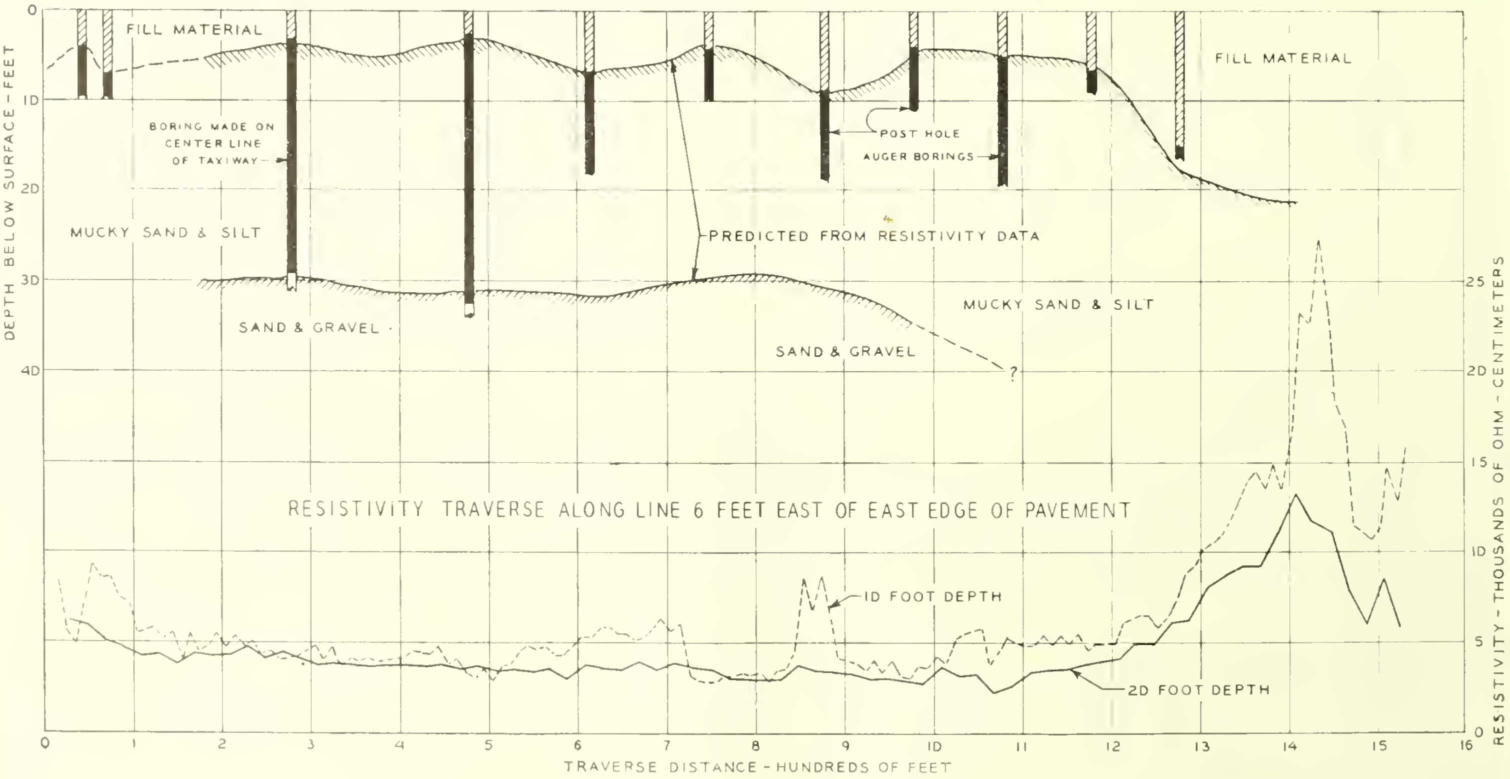
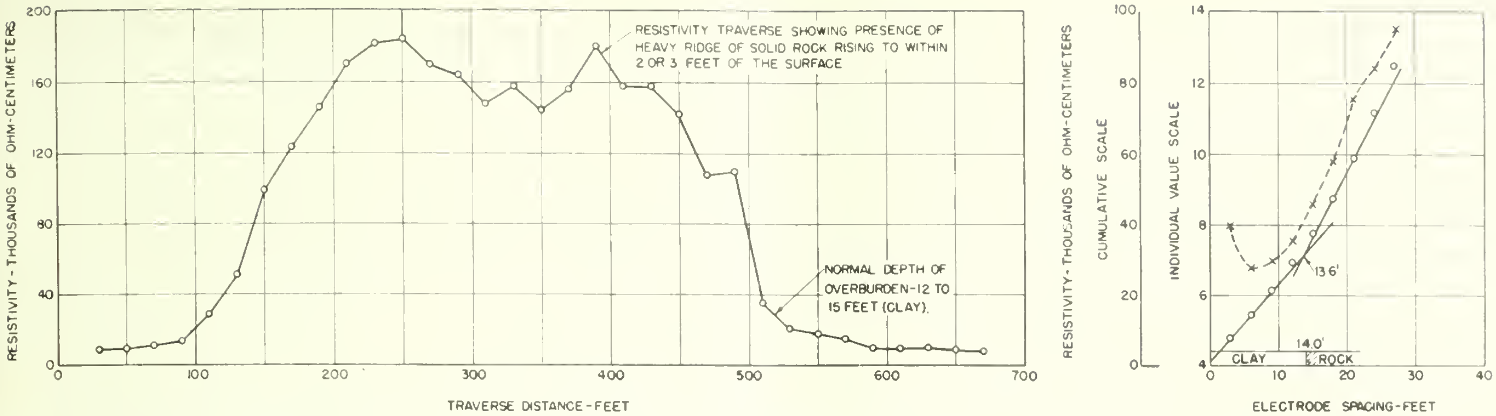


Figure 23.—Resistivity depth tests over peat-bog formations.

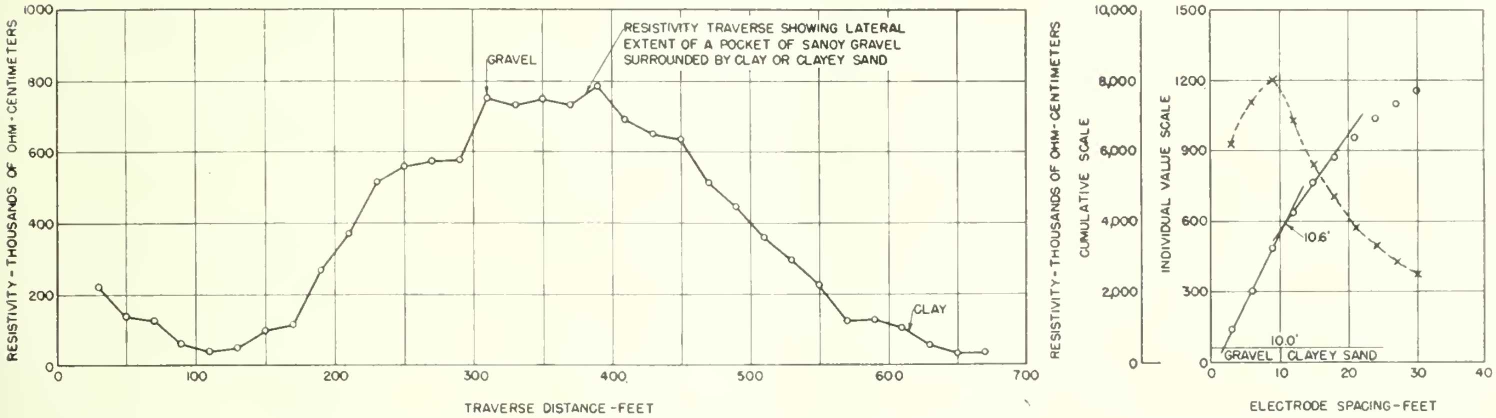
Figure 24 (below).—Results of earth-resistivity tests along the edge of a taxiway at the Washington National Airport.





RESISTIVITY TRAVERSE OVER A BURIED GRANITE RIDGE USING A CONSTANT DEPTH OF 20 FEET.

DEPTH TEST INVOLVING CLAY UNDERLAIN BY SOLID ROCK



RESISTIVITY TRAVERSE OVER A SAND AND GRAVEL DEPOSIT USING A CONSTANT DEPTH OF 20 FEET.

DEPTH TEST INVOLVING SANDY GRAVEL UNDERLAIN BY CLAY OR CLAYEY SAND.

Figure 25.—Resistivity traverses over a rock ridge and a sand and gravel deposit are similar in appearance, but depth tests and a general knowledge of local geology offer some clue as to the actual material involved.

leaving pinnacles and deep valleys in otherwise hard rock (a condition sometimes encountered in limestone formations), the resistivity test may possibly prove the more valuable of the two methods. In such cases the sharp irregularities of the rock surface present unfavorable conditions for consistent interpretation of the seismic data (*24*). To the writer's knowledge, there has been no report on results of resistivity tests carried out in such areas.

The use of explosives as required in the seismic method is not desirable in thickly populated areas. Compliance with local regulations regarding possession and transportation of explosives, sometimes rather strictly enforced, can be troublesome and inconvenient, placing a further handicap upon seismic exploration.

As mentioned previously, the time required for conducting a seismic test can vary from 1 to 3 hours, depending upon local conditions, while resistivity tests can be made at a rate of 3 per hour to depths of 60 feet in rugged mountainous terrain. A seismic party may require one or more men than are necessary for the efficient operation of the resistivity apparatus, particularly in isolated areas

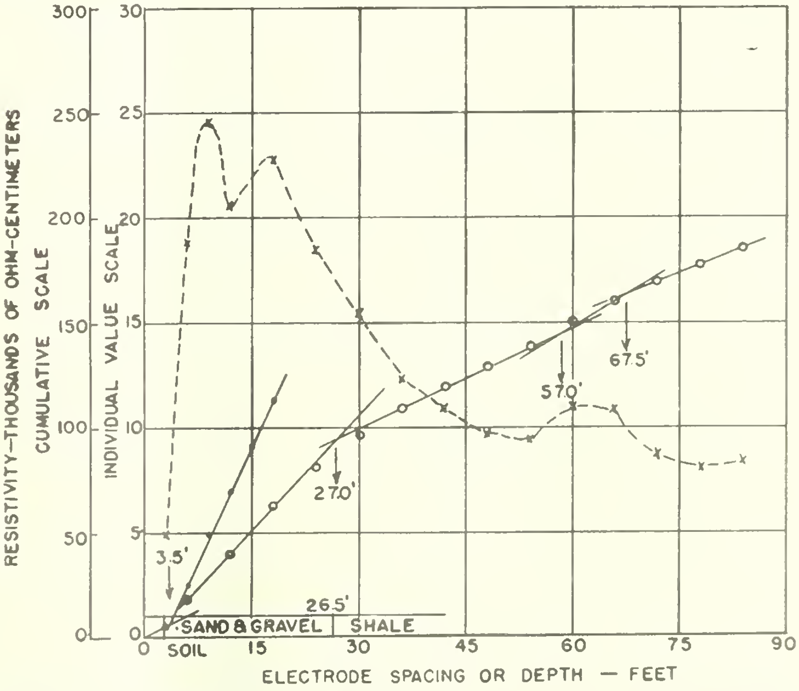


Figure 26.—This resistivity depth test accurately located an underlying shale formation.

where supplies of explosives and film-developing equipment must be carried in by hand. However, stray currents leaving cross-country pipe lines, or emanating from electric railway systems in urban areas, and buried utilities such as water and gas pipes, can be troublesome when making a resistivity survey. These will not affect the efficient use of the seismograph.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF JUNE 30, 1950

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$13,615	$14,192	$7,145	419.7	$4,814	$2,343	156.7	$12,137	$6,295	269.1	$31,143	$15,783	845.5
Arizona	1,425	4,147	2,907	91.5	1,390	971	32.9	5,869	4,201	116.1	11,406	8,079	240.5
Arkansas	1,848	11,112	6,217	287.9	10,768	5,285	305.1	14,027	6,903	323.6	35,907	18,405	916.6
California	3,188	36,484	14,443	238.9	8,499	3,806	86.3	38,683	19,171	217.7	83,666	37,420	542.9
Colorado	3,026	5,603	3,107	128.7	2,919	1,716	96.3	15,611	9,020	300.9	24,133	13,843	525.9
Connecticut	2,323	10,434	5,050	25.3	2,024	982	7.1	6,564	3,797	8.3	19,022	9,829	40.7
Delaware	1,233	2,642	1,330	40.5	2,310	1,152	15.7	4,997	2,371	58.5	9,949	4,853	114.7
Florida	4,826	16,165	8,245	451.7	6,061	3,372	126.9	10,616	5,385	267.2	32,842	17,002	845.8
Georgia	4,139	15,903	8,259	417.6	6,425	3,212	191.0	41,454	19,684	731.8	63,782	31,155	1,340.4
Idaho	4,927	8,920	5,595	340.7	1,793	1,107	46.2	6,604	4,127	179.6	17,317	10,829	566.5
Illinois	18,808	47,642	24,815	367.4	20,409	10,184	166.8	48,597	23,364	353.7	116,648	58,363	887.9
Indiana	16,530	17,986	9,493	66.9	7,524	3,777	64.0	12,768	6,669	55.9	38,278	19,939	186.8
Iowa	3,283	13,397	5,958	607.1	6,834	2,654	351.7	20,595	9,997	758.9	40,826	18,609	1,717.7
Kansas	4,478	11,785	6,035	1,149.1	4,895	2,478	484.2	14,586	7,501	594.5	31,266	16,014	2,227.8
Kentucky	2,460	14,763	7,241	200.7	9,330	4,638	168.7	15,238	7,518	289.2	39,331	19,397	658.6
Louisiana	5,769	19,950	9,665	211.9	8,388	4,106	110.2	16,585	8,562	183.5	44,923	22,333	505.6
Maine	1,857	9,962	5,359	127.7	1,277	715	9.5	5,872	2,911	80.6	17,111	8,985	217.8
Maryland	2,007	7,577	3,656	29.0	2,921	1,199	27.3	17,007	8,211	61.4	27,505	13,066	117.7
Massachusetts	2,314	14,577	6,710	15.5	6,216	3,178	7.5	58,111	28,140	58.7	78,904	38,028	81.7
Michigan	6,969	17,459	9,032	599.6	9,526	4,754	264.0	43,284	17,877	203.6	70,269	31,663	1,067.2
Minnesota	2,370	12,049	6,356	1,124.3	11,154	5,796	761.6	19,902	10,484	513.1	43,105	22,636	2,399.0
Mississippi	10,938	5,824	2,924	301.2	2,871	1,436	130.2	6,183	3,249	196.9	14,878	7,609	628.3
Missouri	7,162	29,740	15,733	848.4	11,410	5,055	279.5	30,440	15,177	605.8	71,590	35,965	1,733.7
Montana	5,320	16,392	8,565	610.7	4,055	2,502	148.9	14,403	8,917	374.2	34,850	19,984	1,133.8
Nebraska	5,298	19,133	9,914	664.2	4,132	1,945	74.7	12,844	7,369	310.1	36,109	19,228	1,049.0
Nevada	1,637	5,474	4,514	190.1	221	180	3.2	5,330	4,366	188.4	11,025	9,060	381.7
New Hampshire	1,774	5,948	2,961	58.5	500	230	2.9	3,798	1,835	27.9	10,246	5,026	89.3
New Jersey	2,560	4,504	2,152	11.5	9,879	4,819	12.3	17,266	8,286	26.7	31,649	15,257	50.5
New Mexico	1,316	9,819	6,264	323.7	2,608	1,692	105.5	6,857	4,468	215.5	19,284	12,424	644.7
New York	34,159	72,813	37,264	231.4	23,018	10,349	43.7	92,013	45,635	185.6	187,844	93,248	460.7
North Carolina	3,460	22,659	10,893	571.0	5,962	2,854	182.8	19,715	9,468	470.9	48,336	23,215	1,224.7
North Dakota	3,942	8,059	4,182	1,390.5	4,401	2,202	273.3	9,714	4,883	672.9	22,174	11,267	2,336.7
Ohio	8,533	50,796	24,012	490.1	23,256	11,927	128.6	41,107	19,920	200.8	115,159	55,859	819.5
Oklahoma	957	22,835	12,527	301.9	11,596	5,374	342.2	14,887	7,267	405.1	49,318	25,168	1,049.2
Oregon	982	7,233	4,194	96.2	1,970	1,152	29.9	13,405	7,604	210.7	22,608	12,950	336.8
Pennsylvania	4,294	38,896	19,087	101.8	17,836	8,869	37.6	73,953	36,545	170.0	130,685	64,501	309.4
Rhode Island	121	9,662	4,918	63.8	1,577	768	2.4	10,172	5,077	7.5	21,411	10,763	73.7
South Carolina	3,104	9,303	4,585	272.1	1,488	797	106.6	12,146	6,084	492.5	22,937	11,466	871.2
South Dakota	1,294	12,632	7,322	1,190.2	4,174	2,533	367.2	7,350	4,503	568.7	24,156	14,358	2,126.1
Tennessee	1,140	15,259	7,393	329.3	11,892	5,466	335.2	18,342	8,506	337.3	45,493	21,365	1,001.8
Texas	9,854	6,223	3,176	428.1	12,296	6,452	552.5	56,277	25,630	1,667.2	74,796	35,258	2,647.8
Utah	1,588	5,179	3,800	177.7	1,590	1,151	62.9	5,677	4,147	119.8	12,446	9,098	360.4
Vermont	1,414	2,721	1,484	40.4	1,548	748	4.6	3,470	1,567	34.5	7,739	3,799	79.5
Virginia	5,502	25,786	12,916	561.4	5,112	2,533	150.7	15,653	7,626	277.3	46,551	23,075	989.4
Washington	1,898	13,256	6,086	132.9	3,329	1,619	98.4	19,100	8,721	126.9	35,685	16,426	358.2
West Virginia	2,146	16,905	7,251	180.7	2,006	1,011	35.9	10,228	5,122	65.1	29,139	13,384	281.7
Wisconsin	8,525	24,911	12,754	473.8	5,471	2,629	242.0	16,252	7,962	384.5	46,634	23,345	1,100.3
Wyoming	850	3,528	2,131	67.3	1,282	808	63.4	7,864	5,184	334.9	12,674	8,123	465.6
Hawaii	1,506	8,036	3,714	24.0	2,187	962	.5	4,190	1,952	21.3	14,413	6,628	45.8
District of Columbia	1,606	5,700	3,063	7.5	119	59	.5	396	198		6,215	3,320	8.0
Puerto Rico	5,615	4,069	1,819	17.0	1,435	662	6.3	10,842	4,575	47.5	16,346	7,056	70.8
TOTAL	245,890	796,044	404,216	17,099.1	314,698	156,209	7,304.1	988,981	494,031	14,372.4	2,099,723	1,054,456	38,775.6

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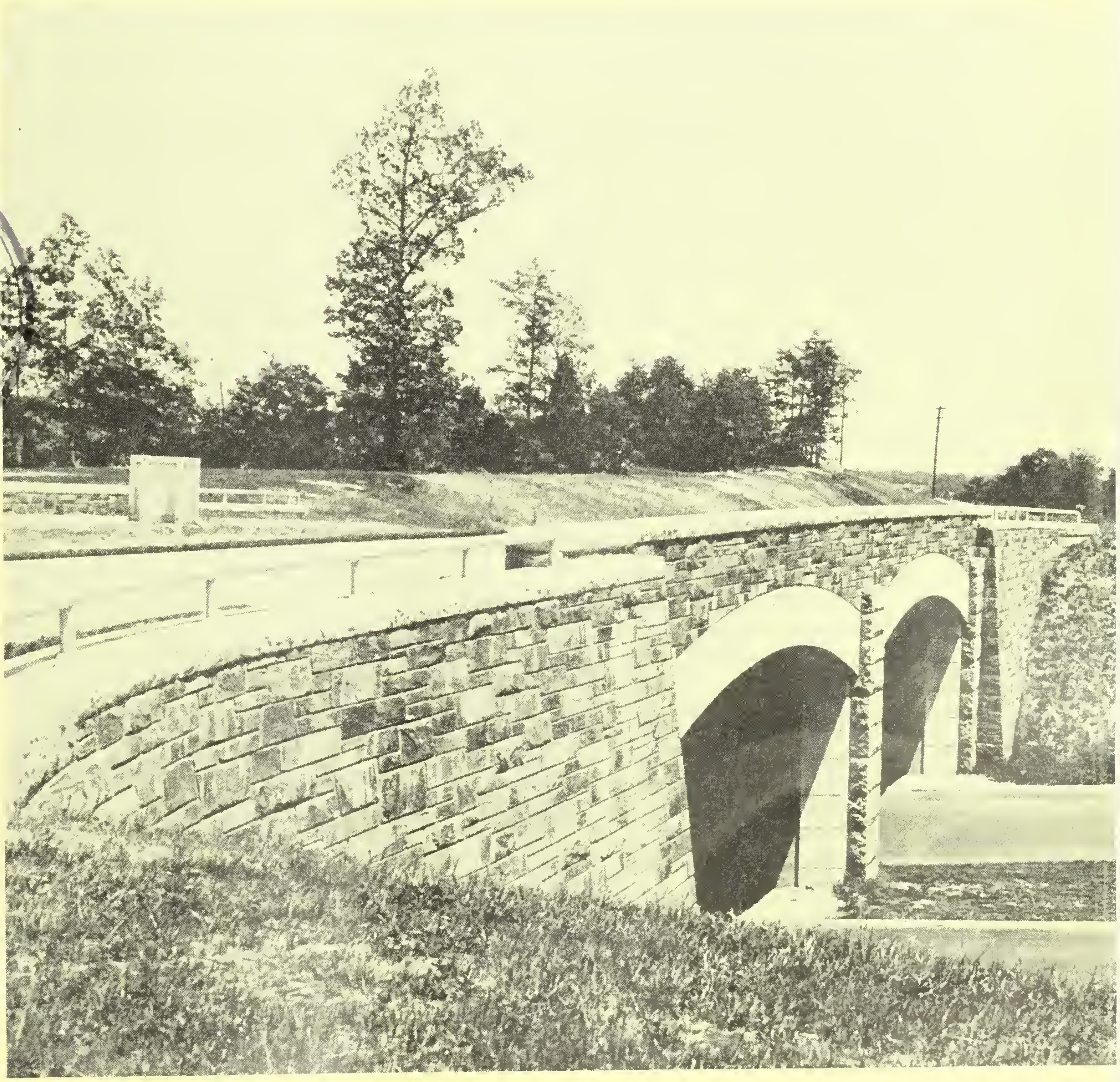

Grade separation on access road to
Andrews Air Field, Maryland

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U. S. DEPARTMENT OF COMMERCE

E. A. STROMBERG, Editor

Moment Distribution Analysis of Two-Span Arched Frames With Elastic Pier

By THOMAS P. REVELISE, [1] **Highway Bridge Engineer, Bureau of Public Roads**

During recent years multiple arches in continuous series have assumed considerable importance in the structural engineering field, and several methods of analysis of such structures by moment and thrust distribution have been developed or proposed. This article presents an adaptation of the method of moment distribution to the analysis of two-span arched frames with elastic center pier and with either fixed or hinged footings. The method is applicable to any two-span continuous arch, but is arranged particularly for convenience in the analysis of arched frames because of the increasingly frequent use of this type of structure for grade separations of divided highways.

Part I of the article is devoted to the necessary mathematical development for a structure with hinged footings, a discussion of procedure, and an actual sample analysis of an unsymmetrical two-span frame with hinged footings. In part II expressions for a structure with fixed footings are developed, followed by a discussion of procedure and a sample analysis of the same structure as that used in part I, but with footings fixed.

The use of forms for tabulating computations makes most of the analysis procedure a mechanical operation by which results can be obtained rapidly and accurately by designers of limited experience.

INTRODUCTION

DURING recent years multiple arches in continuous series have assumed considerable importance in the structural engineering field, and several methods of analysis of such structures by moment and thrust distribution have been developed or proposed.[2]

This paper presents an adaptation of the method of moment distribution to the analysis of two-span arched frames with elastic center pier and with either fixed or hinged footings. The method is applicable to any two-span continuous arch, but is arranged for convenience in the analysis of arched frames due to the importance of this type in divided highway overcrossings. Detailed analyses of an unsymmetrical structure with hinged footings, and of the same structure with fixed footings, illustrate the procedure and facilitate its use in the design office with a minimum of preliminary study of the text or reference to other sources.

Criterion for Arch Analysis

It is first desirable to establish a criterion of deck curvature in order to differentiate arched frames requiring an arch analysis from those that may be analyzed as straight frames with empirical corrections for the effect of arch action. Investigations of this subject based on the application of both methods to a number of typical structures show that when the rise of the deck neutral axis line exceeds approximately one twenty-fifth of the design span, commonly used empirical formulas are not valid. An example of the sensitivity of frames to deck arching is the case of a single-span frame subjected to balanced earth pressure. Under this loading condition a straight frame develops negative moment at the haunch, while a frame identical in every respect except for a deck curvature exceeding the span-rise ratio of 1 to 25 develops a positive moment at the haunch.

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[1] Acknowledgment is made to Dudley P. Babcock and T. P. Weston Jr., Highway Bridge Engineers, for checking the computations and for many helpful suggestions and criticisms.

[2] (1) *Continuous Frames of Reinforced Concrete*, by Hardy Cross and Newlin Morgan; John Wiley & Sons, 1932. (2) Discussion by Donald E. Larson of the paper *Analysis of Continuous Frames by Distributing Fixed-End Moment*, by Hardy Cross; p. 127, Transactions of the American Society of Civil Engineers, vol. 96, 1932. (3) *Analysis of Multiple Arches*, by Alexander Hrennikoff; p. 388, Transactions of the American Society of Civil Engineers, vol. 101, 1936.

If a structure is sufficiently arched to develop fairly pronounced arch action, failure to investigate it as an arch may result in error as to the character of the moments as well as to their magnitude. A ratio of design rise to design span of 1 to 25 is therefore recommended as the criterion that should govern the decision whether or not analysis as a true arch is necessary.

Hinged or Fixed Footings

Most bridge frames are founded on material of yielding character and are designed on the assumption of hinged conditions at the footings. The footings may be constructed integrally with the pier and abutment stems, or separated by some device such as lead plates to reduce the degree of fixity at the base.

Occasionally the structure is founded on rock. Full fixity at the footings is assumed in the design in this case since the bases of the pier and abutment stems are usually imbedded in the rock and the excavations made for that purpose are filled with concrete. It is recognized that ideal conditions of restraint are practically unattainable and that the actual condition for most structures is intermediate between hinged and fully fixed. The usual practice, nevertheless, is to base the design on either an ideal hinged condition or an ideal fixed condition, giving due consideration to the character of the foundation and type of footing to be constructed.

Design constants and forms for tabulating computations are developed in this paper for both hinged and fixed footings. In general, the variation in the two procedures is analagous to that which is encountered in the application of ordinary moment distribution to straight-framed structures having hinged and fixed members.

Part I of the paper is devoted to the necessary mathematical development for a hinged condition, a discussion of procedure, and an actual sample analysis of an unsymmetrical two-span frame with hinged footings. In Part II expressions for a fixed condition are developed, followed by a discussion of procedure and a sample analysis of the same structure used in Part I, but with footings fixed.

Steps in the Analysis

In deriving the design constants, the frame leg and contiguous arched deck are treated as a structural unit. A load of unity is placed at 10 points on each arch, and fixed-end moments and thrusts are computed at the juncture of the deck members and pier. The fixed-end moments and thrusts are then distributed at this joint until the desired convergence is reached.

The first step of the analysis, computation of fixed-end moments and thrusts for various positions of a unit load, constitutes solution of a single-span unsymmetrical arch, fixed at the connection with the pier and either hinged or fixed at the footing. Formulas for this computation are derived from the basic elastic equations of rotation and displacement. The resultant expressions are adapted to a form for tabulating computations in which computed values of moment, M, vertical reaction, V, and horizontal thrust, H, are obtained directly at the points of fixity for 10 positions of a unit load on each arch. No sketching of influence lines is necessary. By using unit values in the distribution procedure, only two distributions are required for symmetrical structures, and four if the structure is unsymmetrical. The necessary joint constants are evaluated from expressions derived in the computation for fixed-end moments and thrusts. It is recommended that the computations be made on a calculator, and with a degree of accuracy not less than that indicated in the sample analyses.

After the indeterminate moments and reactions are obtained, further design data may be derived in the same manner as for statically determinate structures. This portion of the work is subject to considerable variation and is omitted in the sample analyses.

Tabulating Forms Used

The use of forms for tabulating computations renders most of the procedure mechanical in nature. Experience with similar forms in arch and arched-frame analysis shows that results can be obtained rapidly and accurately by designers of limited experience. The method is thus applicable directly, without reference to the mathematical derivations.

The analysis of two-span arches and arched frames with elastic pier is especially well adapted to the type of procedure illustrated in the sample analyses. The various operations are, in general, analogous to the procedure of ordinary moment distribution, and the convergence of values is rapid. In view of the comparatively limited variation in the geometric characteristics of this type of structure, it is doubtful that actual cases will occur in which the conversion of the distribution cycles is retarded to an objectionable degree.

Occasionally special architectural treatment of the structure, such as the addition of stone facing, results in a pier of sufficient mass to virtually break flexural continuity at the joint. In such cases it is logical to assume fixity at the joint, and design each arch independently.

Somewhat greater refinement in the computed values could be obtained by placing the unit loads between, instead of at the elastic centers of gravity. It is believed, however, that the accuracy of the method used is not inconsistent with the limitations imposed by uncertainties in basic assumptions and in construction.

Following part II of the paper there appears a series of influence lines (figs. 13 and 14, p. 84), showing the comparative effect of hinged and fixed footings at some of the critical points.

Part I.—ANALYSIS OF STRUCTURES WITH HINGED FOOTINGS

Required Design Constants

Three structural elements are considered in the analysis, as illustrated in figure 1: Unsymmetrical arch AB, unsymmetrical arch BC, and elastic pier BD. The required design constants are defined as follows:

Fixed-end moment.—The moment at B due to a load P or P', if B were completely fixed.

Fixed-end thrust.—The thrust at A or C due to a load P or P', if B were completely fixed.

Moment stiffness of arch rib.—The moment at B necessary to produce a rotation of unity, without horizontal or vertical displacement at B.

Induced thrust of arch rib.—The thrust at A or C induced by a moment of unity at B, without horizontal or vertical displacement at B.

Thrust stiffness of arch rib.—The thrust at B necessary to produce a horizontal displacement of unity at B without vertical displacement or rotation at B.

Induced moment of arch rib.—The moment at B induced by a horizontal thrust of unity at A or C, without vertical displacement or rotation at B.

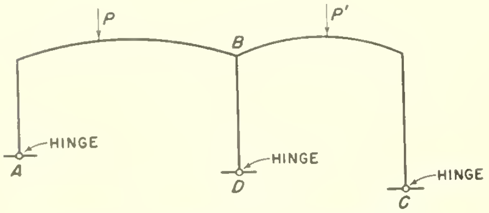


Figure 1.—Design sketch of frame.

Development of Fixed-End Moment and Thrust

In figure 2 the arch rib from the hinge at the footing A to the juncture with the elastic pier and adjacent arch rib at B is treated as a single structural member, fixed at B. The term "arch rib" is used in referring to these members because of the common practice of basing barrel-arch analyses on an element 1 foot wide. This practice is followed in the sample analysis. A concentrated load P is placed on the arch rib, inducing a fixed-end moment at B and equal fixed-end thrusts at B and A. Both the point at which the load P is applied, and the horizontal distance from the neutral axis of the frame leg to that point, are represented as a. In usage, it will be obvious whether a represents the point or the distance.

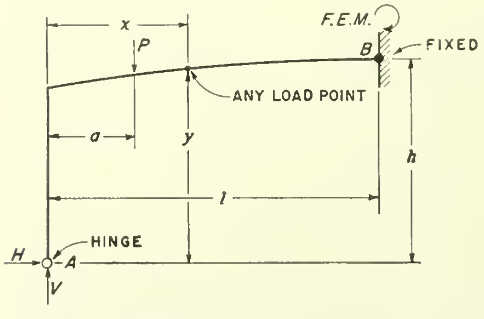


Figure 2.—Sketch for use in deriving fixed-end moment and thrust.

The moment from A to $a = Vx - Hy$.

The moment from a to $B = Vx - Hy - P(x-a)$.

Letting P equal unity, the moment from a to $B = Vx - Hy - (x-a)$.

From the condition that point A is not displaced horizontally and from the condition that point A is not displaced vertically the following two elastic equations may be written:

$$\sum_{A}^{B} \frac{Mxds}{EI} \text{ (vertical displacement)} = 0 \quad (1)$$

$$\sum_{A}^{B} \frac{Myds}{EI} \text{ (horizontal displacement)} = 0 \quad (2)$$

The modulus of elasticity, E, is constant for the entire structure and may be eliminated from the basic equations in evaluating external reactions due to flexure. In addition, it will simplify the expressions somewhat to use a single symbol for values of ds/I. Accordingly, ds/I is represented by the symbol Δ.

Making these modifications, equations (*1*) and (*2*) may be restated as follows:

$$\sum_{A}^{B} Mx\Delta = 0 \quad (3)$$

$$\sum_{A}^{B} My\Delta = 0 \quad (4)$$

Inserting the general expression for moment into equations (*3*) and (*4*), the following are obtained:

$$V \sum_{A}^{B} x^2\Delta - H \sum_{A}^{B} xy\Delta - \sum_{a}^{B} (x-a)x\Delta = 0 \quad (5)$$

$$V \sum_{A}^{B} xy\Delta - H \sum_{A}^{B} y^2\Delta - \sum_{a}^{B} (x-a)y\Delta = 0 \quad (6)$$

Solving equations (*5*) and (*6*) for H and V:

$$H = \frac{\sum_{a}^{B} (x-a)x\Delta \sum_{A}^{B} xy\Delta - \sum_{a}^{B} (x-a)y\Delta \sum_{A}^{B} x^2\Delta}{\sum_{A}^{B} x^2\Delta \sum_{A}^{B} y^2\Delta - \left(\sum_{A}^{B} xy\Delta\right)^2} \quad (7)$$

$$V = \frac{\sum_{a}^{B} (x-a)x\Delta \sum_{A}^{B} y^2\Delta - \sum_{a}^{B} (x-a)y\Delta \sum_{A}^{B} xy\Delta}{\sum_{A}^{B} x^2\Delta \sum_{A}^{B} y^2\Delta - \left(\sum_{A}^{B} xy\Delta\right)^2} \quad (8)$$

When x is less than a, the value of the term $(x-a)$ is zero in equations (*5*), (*6*), (*7*), and (*8*).

The work entailed in evaluating H and V is substantially reduced by a modification of equations (*7*) and (*8*). The assumption is made that the $x\Delta$ and $y\Delta$ values represent actual loads, concentrated at the midpoints of equal dx divisions. Making this assumption, the expressions

$$\sum_{a}^{B} (x-a)x\Delta \text{ and } \sum_{a}^{B} (x-a)y\Delta$$

in equations (*7*) and (*8*) then represent the cantilever moments about point a of the $x\Delta$ loads and $y\Delta$ loads between a and B. These cantilever moments about point a may in turn be expressed as the areas of the $x\Delta$ and $y\Delta$ shear diagrams between a and B. The ordinate of the $x\Delta$ shear diagram at any load point between a and B is:

$$\sum_{a+1}^{B} x\Delta$$

In this expression a is any load point between the load and B. The summation is taken from $a+1$ (a plus one load point) because the $x\Delta$ concentration at point a theoretically passes through the assumed point of support and thus causes no shear.

Having developed an expression for the ordinate of the $x\Delta$ shear diagram at any load point between a and B, it is apparent that the area of the shear diagram may be expressed as the sum of the ordinates at the centers of the equal dx divisions multiplied by the length of those divisions. Hence the $x\Delta$ cantilever moment and the $y\Delta$ cantilever moment expressions, in both of which a, in the first summation symbol, is the point of load, are:

$$x\Delta \text{ cantilever moment} = \left(\sum_{a}^{B} \sum_{a+1}^{B} x\Delta\right) dx$$

$$y\Delta \text{ cantilever moment} = \left(\sum_{a}^{B} \sum_{a+1}^{B} y\Delta\right) dx$$

The above expressions are equated respectively to the terms

$$\sum_{a}^{B} (x-a)x\Delta \text{ and } \sum_{a}^{B} (x-a)y\Delta$$

and are substituted in equations (*7*) and (*8*), which now become:

$$H = \frac{\left(\sum_{a}^{B} \sum_{a+1}^{B} x\Delta \sum_{A}^{B} xy\Delta - \sum_{a}^{B} \sum_{a+1}^{B} y\Delta \sum_{A}^{B} x^2\Delta\right) dx}{\sum_{A}^{B} x^2\Delta \sum_{A}^{B} y^2\Delta - \left(\sum_{A}^{B} xy\Delta\right)^2} \quad (9)$$

$$V = \frac{\left(\sum_{a}^{B} \sum_{a+1}^{B} x\Delta \sum_{A}^{B} y^2\Delta - \sum_{a}^{B} \sum_{a+1}^{B} y\Delta \sum_{A}^{B} xy\Delta\right) dx}{\sum_{A}^{B} x^2\Delta \sum_{A}^{B} y^2\Delta - \left(\sum_{A}^{B} xy\Delta\right)^2} \quad (10)$$

For temperature change:

$$H_T = \pm \frac{(ETle)\left(\sum_{A}^{B} x^2\Delta\right)}{\sum_{A}^{B} x^2\Delta \sum_{A}^{B} y^2\Delta - \left(\sum_{A}^{B} xy\Delta\right)^2} \quad (11)$$

$$V_T = \pm \frac{(ETle)\left(\sum_{A}^{B} xy\Delta\right)}{\sum_{A}^{B} x^2\Delta \sum_{A}^{B} y^2\Delta - \left(\sum_{A}^{B} xy\Delta\right)^2} \quad (12)$$

in which

E = modulus of elasticity.
T = number of degrees change in temperature.
l = design span length.
e = coefficient of expansion.

It is customary to use the plus sign to designate values of H_T and V_T caused by a rise in temperature.

Equations (*9*), (*10*), (*11*), and (*12*) may now be used for evaluation of fixed-end moments and fixed-end thrusts due to concentrated loads and temperature change, as shown in the sample analysis.

Development of Arch Rib Design Constants

In figure 3 the hinge at A is assumed to be cut free, and joint B given a rotation α. Joint B is is then locked in this position and A is returned to its original position by first a horizontal displacement, $h\alpha$, without vertical displacement, and then by a vertical displacement, $l\alpha$, without horizontal displacement. The moment induced at B is that which would have been induced by holding A in its original position against displacement and rotating B through the angle α. The moment stiffness of the arch rib is expressed by the term M/α in which M is the moment at B induced by the rotation α.

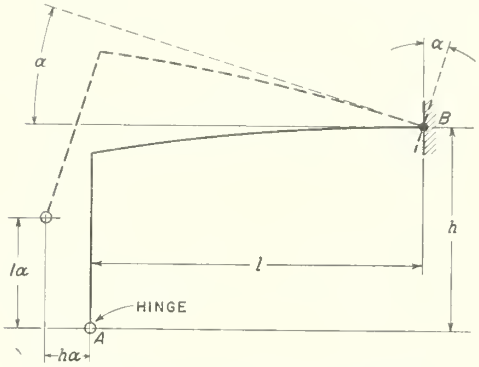


Figure 3.—Sketch for use in deriving moment stiffness of arch rib.

The procedure is performed in two steps, as indicated in the previous paragraph. In step 1, A is given a horizontal displacement, $h\alpha$, without vertical displacement, and H_1, V_1, and M_{1B} are derived. In step 2, A is given a vertical displacement, $l\alpha$, without horizontal displacement, and H_2, V_2, and M_{2B} are derived. The final moment at B is thus $M_{1B} + M_{2B}$ and the final thrust is $H_1 + H_2$. These correlated values may be used in obtaining the mathematical expressions for the previously defined arch rib design constants as follows:

Moment stiffness of arch rib $= (M_{1B} + M_{2B}) \div \alpha$
Induced thrust of arch rib $= (H_1 - H_2) \div (M_{1B} + M_{2B})$
Thrust stiffness of arch rib $= H_1 + h\alpha$
Induced moment of arch rib $= M_{1B} \div H_1$

Step 1.—In deriving expressions for fixed-end moment and fixed-end thrust the modulus of elasticity, E, is omitted and the recurrent

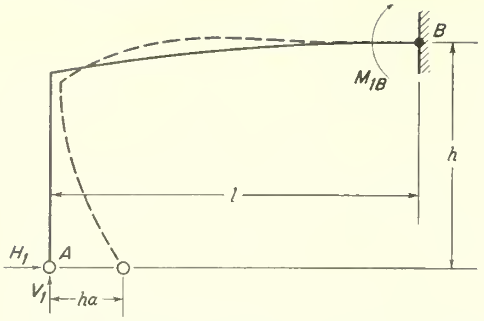


Figure 4.—Sketch for use in step 1 derivations.

term ds/I is represented by the symbol Δ for convenience. The absolute expressions for moment and thrust stiffness, however, contain E as a function. Therefore, the term Δ/E is used in deriving these constants.

From the condition that A is displaced horizontally a distance $h\alpha$ and from the condition that A is not displaced vertically, as shown in figure 4, the following two elastic equations may be written:

$$\sum_A^B My\frac{\Delta}{E}=h\alpha\ldots\ldots(13)$$

$$\sum_A^B Mx\frac{\Delta}{E}=0\ldots\ldots(14)$$

No sign is affixed to the displacement term, $h\alpha$, since the direction of the displacement and of the forces causing it is obvious, as noted in figure 4.

The moment at any point between A and $B=V_1x-H_1y$. Inserting the general expression for moment into equations (*13*) and (*14*) the following are obtained:

$$V_1\sum_A^B xy\frac{\Delta}{E}-H_1\sum_A^B y^2\frac{\Delta}{E}=h\alpha\ldots\ldots(15)$$

$$V_1\sum_A^B x^2\frac{\Delta}{E}-H_1\sum_A^B xy\frac{\Delta}{E}=0\ldots\ldots(16)$$

Solving equations (*15*) and (*16*) for H_1 and V_1:

$$H_1=\left[\frac{h\alpha\sum_A^B x^2\Delta}{\left(\sum_A^B xy\Delta\right)^2-\sum_A^B x^2\Delta\sum_A^B y^2\Delta}\right]E\ldots(17)$$

$$V_1=\left[\frac{h\alpha\sum_A^B xy\Delta}{\left(\sum_A^B xy\Delta\right)^2-\sum_A^B x^2\Delta\sum_A^B y^2\Delta}\right]E\ldots(18)$$

Whence:

$$M_{1B}=V_1l-H_1h=$$

$$\left[\frac{l\left(h\alpha\sum_A^B xy\Delta\right)-h\left(h\alpha\sum_A^B x^2\Delta\right)}{\left(\sum_A^B xy\Delta\right)^2-\sum_A^B x^2\Delta\sum_A^B y^2\Delta}\right]E\ldots(19)$$

Step 2.—From the condition that A is displaced a distance $l\alpha$ and from the condition that A is not displaced horizontally, as shown in figure 5, the following elastic equations may be written:

$$\sum_A^B Mx\frac{\Delta}{E}=l\alpha\ldots\ldots(20)$$

$$\sum_A^B My\frac{\Delta}{E}=0\ldots\ldots(21)$$

The moment at any point between A and $B=H_2y-V_2x$. Inserting the general expression for moment into equations (*20*) and (*21*) the following are obtained:

$$H_2\sum_A^B xy\frac{\Delta}{E}-V_2\sum_A^B x^2\frac{\Delta}{E}=l\alpha\ldots\ldots(22)$$

$$H_2\sum_A^B y^2\frac{\Delta}{E}-V_2\sum_A^B xy\frac{\Delta}{E}=0\ldots\ldots(23)$$

Solving equations (*22*) and (*23*) for H_2 and V_2:

$$H_2=\left[\frac{l\alpha\sum_A^B xy\Delta}{\left(\sum_A^B xy\Delta\right)^2-\sum_A^B x^2\Delta\sum_A^B y^2\Delta}\right]E\ldots(24)$$

$$V_2=\left[\frac{l\alpha\sum_A^B y^2\Delta}{\left(\sum_A^B xy\Delta\right)^2-\sum_A^B x^2\Delta\sum_A^B y^2\Delta}\right]E\ldots(25)$$

Whence:

$$M_{2B}=H_2h-V_2l=$$

$$\left[\frac{h\left(l\alpha\sum_A^B xy\Delta\right)-l\left(l\alpha\sum_A^B y^2\Delta\right)}{\left(\sum_A^B xy\Delta\right)^2-\sum_A^B x^2\Delta\sum_A^B y^2\Delta}\right]E\ldots(26)$$

The basic expressions for the arch rib design constants have already been stated. Substituting in these the expressions for H_1, V_1, and M_{1B}, derived as equations (*17*), (*18*), and (*19*) in step 1, and the expressions for H_2, V_2, and M_{2B}, derived as equations (*24*), (*25*), and (*26*) in step 2, the arch rib design constants may now be expressed as follows:

Moment stiffness of arch rib$=$
$(M_{1B}+M_{2B})\div\alpha=$

$$\left\{\left[\frac{l\left(h\alpha\sum_A^B xy\Delta-l\alpha\sum_A^B y^2\Delta\right)+h\left(l\alpha\sum_A^B xy\Delta-h\alpha\sum_A^B x^2\Delta\right)}{\left(\sum_A^B xy\Delta\right)^2-\sum_A^B x^2\Delta\sum_A^B y^2\Delta}\right]E\right\}\div\alpha=$$

$$\left[\frac{l\left(h\sum_A^B xy\Delta-l\sum_A^B y^2\Delta\right)+h\left(l\sum_A^B xy\Delta-h\sum_A^B x^2\Delta\right)}{\left(\sum_A^B xy\Delta\right)^2-\sum_A^B x^2\Delta\sum_A^B y^2\Delta}\right]E\ldots\ldots(27)$$

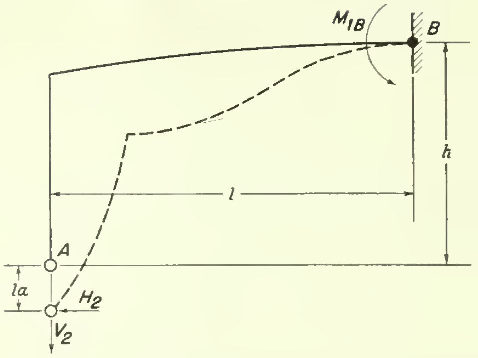


Figure 5.—Sketch for use in step 2 derivations.

Induced thrust of arch rib$=$
$(H_1-H_2)\div(M_{1B}+M_{2B})=$

$$\frac{h\sum_A^B x^2\Delta-l\sum_A^B xy\Delta}{l\left(h\sum_A^B xy\Delta-l\sum_A^B y^2\Delta\right)+h\left(l\sum_A^B xy\Delta-h\sum_A^B x^2\Delta\right)}\quad(28)$$

Thrust stiffness of arch rib$=H_1\div h\alpha=$

$$\left[\frac{\sum_A^B x^2\Delta}{\left(\sum_A^B xy\Delta\right)^2-\sum_A^B x^2\Delta\sum_A^B y^2\Delta}\right]E\ldots\ldots(29)$$

Induced moment of arch rib $=M_{1B}\div H_1=$

$$\frac{l\sum_A^B xy\Delta-h\sum_A^B x^2\Delta}{\sum_A^B x^2\Delta}\ldots\ldots(30)$$

Development of Elastic Pier Constants

The elastic pier is assumed to be of uniform cross section, in accordance with common construction practice. The moment of inertia of this member is constant, therefore, and the expressions for moment stiffness, thrust stiffness, induced moment, and induced thrust can be derived directly without recourse to the summation process which is required in the case of the arch rib constants. Referring to figure 6:

Moment stiffness of pier$=$

$$\frac{M}{\alpha}=Hl\div\frac{l\alpha}{l}=\frac{(Hl)(3EI)}{Hl^2}=\frac{3EI}{l}\ldots\ldots(31)$$

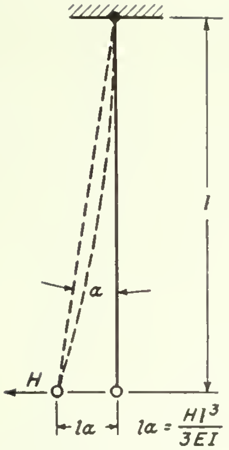


Figure 6.—Sketch for use in deriving pier constants.

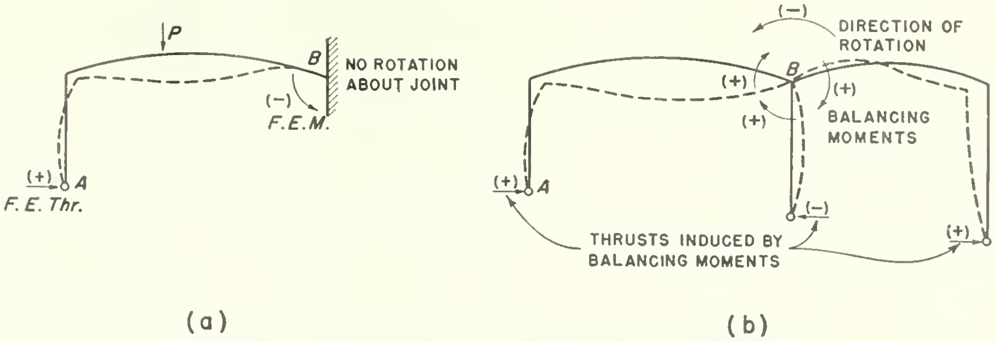


Figure 7.—Sketches for use in deriving sign convention.

Induced thrust of pier=

$$\frac{H}{M}=\frac{H}{Hl}=\frac{1}{l}\text{------------}(32)$$

Thrust stiffness of pier=

$$\frac{H}{l\alpha}=H\div\frac{Hl^3}{3EI}=\frac{3EI}{l^3}\text{--------}(33)$$

Induced moment of pier=

$$\frac{M}{H}=\frac{Hl}{H}=l\text{------------}(34)$$

Derivation of Sign Conventions

The system adopted for indicating the signs of the moments and thrusts is that which seems to be preferred by most designers. Moments tending to cause rotation about the joint in a clockwise direction are given a plus sign; moments which tend to cause rotation about the joint in a counter-clockwise direction are given a minus sign. Thrusts to the right are given a plus sign, and thrusts to the left are given a minus sign.

In figure 7 (a), joint B is locked and a load, P, is imposed on the arch. A moment is induced at B, and a thrust at A. The moment tends to rotate the joint in a counter-clockwise direction and is given a minus sign. The thrust is to the right and is given a plus sign.

Figure 7 (b) illustrates the action during the first moment distribution cycle. When fixity at B is removed, rotation about the joint is toward the imposed load; and the unbalanced fixed-end moment at B is stabilized by induced balancing moments in all of the members forming the joint. These balancing moments are proportional to the moment stiffnesses of the members, their sum exactly equals the fixed-end moment, and they all oppose the counter-clockwise rotation, thereby stabilizing the joint. They are accordingly given a plus sign.

The balancing moments induce thrusts as shown in figure 7 (b). Note that the fixed-end thrust has not been considered in the discussion, and that the thrusts shown in figure 7 (b) are entirely induced by the balancing moments.

These considerations lead to the establishment of the following rules of signs:

Arch ribs: Induced thrusts have the same sign as balancing moments. Induced moments have the same sign as balancing thrusts.

Pier: Induced thrusts have a sign opposite to that of balancing moments. Induced moments have a sign opposite to that of balancing thrusts.

SAMPLE ANALYSIS—I

Two-span arched frame with elastic pier, unsymmetrical both horizontally and vertically, and with hinged footings.

Application of Method

Evaluation of the design constants for arch ribs and pier of a two-span arched frame with elastic pier, unsymmetrical both horizontally and vertically, and with hinged footings, is illustrated in the sample analysis that follows.

Component terms in the expressions derived in the preceding section are entered in tabulation forms and are evaluated individually, and are then recombined to obtain the required final values. For greater clarity and speed, many component terms are referred to on the tabulation forms by column number.

Except in unusual cases the entire structure will be composed of the same structural material. The modulus of elasticity, E, may therefore be omitted in evaluating the stiffness constants, since only the relative stiffnesses are required.

(Text continued on page 72.)

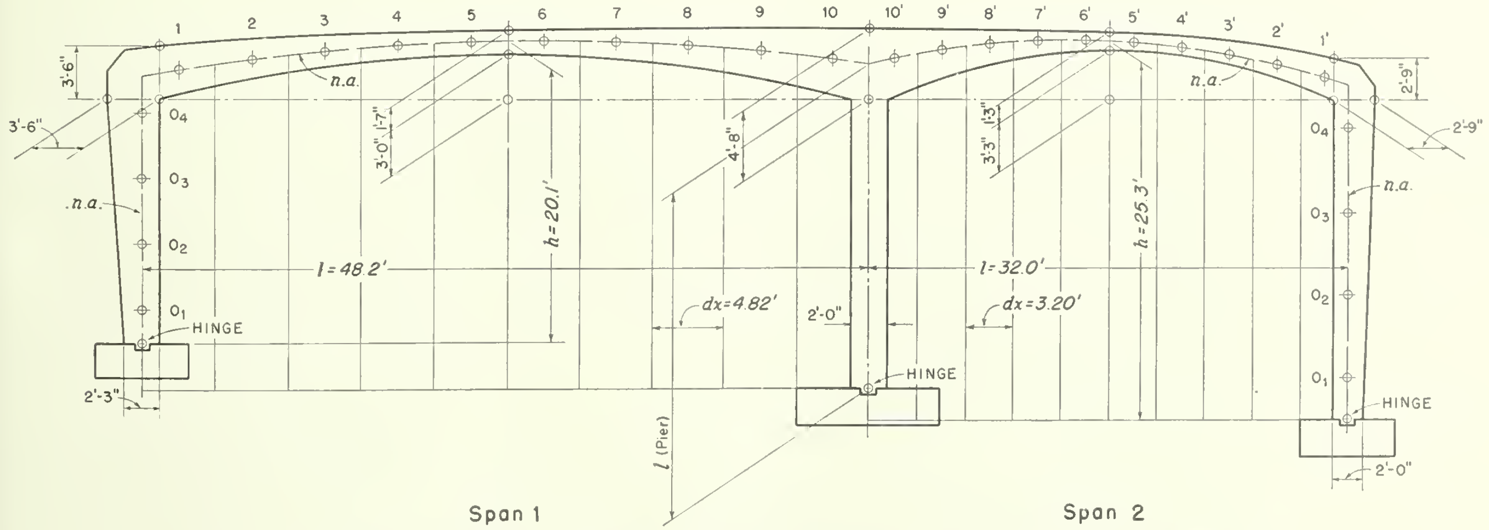


Figure 8.—Working drawing of hinged, unsymmetrical two-span arched frame with elastic pier.

Table 1 (a).—*Fixed-end moments, fixed-end thrusts, and joint constants, span 1*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Point or a	t	(col. 2)^3	ds	x	y	col. 4 ÷ col. 3	col. 5 × col. 7	col. 6 × col. 7	col. 5 × col. 8	col. 6 × col. 9	col. 5 × col. 9	$\sum_{a+1}^{B}$ col. 8	$\sum_{a+1}^{B}$ col. 9	$\sum_{a}^{B}$ col. 13	$\sum_{a}^{B}$ col 14
0_1	2.46	14.887	4.87	0	2.43	0.327	0	0.80	0	2.6	0	0	0	0	0
0_2	2.78	21.485	4.87	0	7.30	.227	0	1.66	0	12.1	0	0	0	0	0
0_3	3.17	31.855	4.87	0	12.17	.153	0	1.86	0	22.6	0	0	0	0	0
0_4	3.45	41.064	4.87	0	17.04	.119	0	2.03	0	34.6	0	0	0	0	0
1	3.33	36.926	4.92	2.41	19.92	.133	.32	2.65	.8	52.8	6.4	130.42	119.55	650.01	520.24
2	2.58	17.174	4.88	7.23	20.58	.284	2.05	5.84	14.8	120.2	42.2	128.37	113.71	519.59	400.69
3	2.04	8.490	4.85	12.05	21.08	.571	6.88	12.04	82.9	253.8	145.1	121.49	101.67	391.22	286.98
4	1.70	4.913	4.81	16.87	21.42	.979	16.52	20.97	278.7	449.2	353.8	104.97	89.70	269.73	185.31
5	1.60	4.096	4.81	21.69	21.60	1.174	25.46	25.36	552.2	547.8	550.1	79.51	55.34	164.76	104.61
6	1.60	4.086	4.81	26.51	21.64	1.174	31.12	25.36	825.0	548.8	672.3	48.39	29.98	85.25	49.27
7	1.83	6.128	4.81	31.33	21.58	.785	24.59	16.94	770.4	365.6	530.7	23.80	13.04	36.86	19.29
8	2.33	12.649	4.85	36.15	21.33	.383	13.85	8.17	500.7	174.3	295.3	9.95	4.87	13.06	6.25
9	3.08	29.218	4.88	40.97	20.92	.167	6.84	3.49	280.2	73.0	143.0	3.11	1.38	3.11	1.38
10	4.17	72.512	4.92	45.79	20.35	.068	3.11	1.38	142.4	28.1	63.2	0	0	0	0
								$\sum=$	3,448	2,686	2,802				

1	17	18	19	20	21	22	23	24	25	26	27
Point or a	$\frac{\sum \text{col. } 12}{1{,}000}$ ×col. 15	$\frac{\sum \text{col. } 10}{1{,}000}$ ×col. 16	$\frac{\sum \text{col. } 11}{1{,}000}$ ×col. 15	$\frac{\sum \text{col. } 12}{1{,}000}$ ×col. 16	(col. 17−col. 18) ×dx	(col. 19−col. 20) ×dx	$H=$ col. 21 ÷C	$V=$ col. 22 ÷C	(col. 24 ×l)−(col. 23 ×h)	$l-$ col. 5	$M_B=$ col. 25−col. 26
0_1	0	0	0	0	0	0	0	0	0	0	0
0_2	0	0	0	0	0	0	0	0	0	0	0
0_3	0	0	0	0	0	0	0	0	0	0	0
0_4	0	0	0	0	0	0	0	0	0	0	0
1	1,821.3	1,793.8	1,745.9	1,457.7	132.5	1,389.1	.094	.985	45.59	45.79	−.20
2	1,455.9	1,381.6	1,395.6	1,122.7	358.1	1,315.4	.254	.933	39.86	40.97	−1.11
3	1,096.2	989.5	1,051.8	804.1	514.3	1,189.1	.365	.843	33.29	36.15	−2.86
4	755.8	638.9	724.5	519.2	563.5	989.5	.400	.702	25.80	31.33	−5.53
5	461.7	360.7	442.5	293.1	486.8	720.1	.345	.511	17.70	26.51	−8.81
6	238.9	169.9	229.0	138.1	332.6	438.1	.236	.311	10.25	21.69	−11.44
7	103.3	66.5	99.0	54.1	177.4	216.4	.126	.153	4.84	16.87	−12.03
8	36.6	21.6	35.1	17.5	72.3	84.8	.051	.060	1.82	12.05	−10.23
9	8.7	4.8	8.4	3.9	18.8	21.7	.013	.015	.46	7.23	−6.77
10	0	0	0	0	0	0	0	0	0	2.41	−2.41

h (arch rib)=20.1 ft. l (arch rib)=48.2 ft. dx=4.82 ft. $C=\frac{\sum \text{col. } 10 \sum \text{col. } 11-(\sum \text{col. } 12)^2}{1{,}000}=1{,}410$

$$\text{Moment stiffness of arch rib}=\frac{l\left(\frac{l\sum \text{col. } 11}{1{,}000}-\frac{h\sum \text{col. } 12}{1{,}000}\right)+h\left(\frac{h\sum \text{col. } 10}{1{,}000}-\frac{l\sum \text{col. } 12}{1{,}000}\right)}{12\ C}=0.130$$

$$\text{Induced thrust of arch rib}=\left(\frac{l\sum \text{col. } 12}{1{,}000}-\frac{h\sum \text{col. } 10}{1{,}000}\right)\div(\text{moment stiffness}\times 12\ C)=0.030$$

$$\text{Thrust stiffness of arch rib}=\frac{\sum \text{col. } 10}{1{,}000}\div 12\ C=0.00020$$

$$\text{Induced moment of arch rib}=\left(l\,\frac{\sum \text{col. } 12}{1{,}000}-h\,\frac{\sum \text{col. } 10}{1{,}000}\right)\div\frac{\sum \text{col. } 10}{1{,}000}=19.1$$

$$H_T=\frac{ETle\times\frac{\sum \text{col. } 10}{1{,}000}}{12\ C} \qquad V_T=\frac{ETle\times\frac{\sum \text{col. } 12}{1{,}000}}{12\ C}$$

Table 1 (b).—*Fixed-end moments, fixed-end thrusts, and joint constants, span 2*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Point or a	t	(col. 2) ^{3}	ds	x	y	col. 4 ÷ col. 3	col. 5 × col. 7	col. 6 × col. 7	col. 5 × col. 8	col. 6 × col. 9	col. 5 × col. 9	$\sum_{a+1}^{B}$ col. 8	$\sum_{a+1}^{B}$ col. 9	$\sum_{a}^{B}$ col. 13	$\sum_{a}^{B}$ col. 14
0_1	2.09	9.129	6.00	0	3.00	0.657	0	1.97	0	5.9	0	0	0	0	0
0_2	2.27	11.697	6.00	0	9.00	.513	0	4.62	0	41.6	0	0	0	0	0
0_3	2.45	14.706	6.00	0	15.00	.408	0	6.12	0	91.8	0	0	0	0	0
0_4	2.63	18.191	6.00	0	21.00	.330	0	6.93	0	145.5	0	0	0	0	0
1′	2.50	15.625	3.33	1.60	24.51	.213	.34	5.23	.5	128.3	8.4	86.93	159.53	405.93	645.28
2′	1.96	7.530	3.27	4.80	25.40	.434	2.08	11.02	10.0	279.9	52.9	84.85	148.51	319.00	485.75
3′	1.62	4.252	3.25	8.00	26.04	.764	6.11	19.89	48.9	517.9	159.1	78.74	118.62	234.15	337.24
4′	1.42	2.863	3.23	11.20	26.60	1.128	12.63	30.00	141.5	798.0	336.0	66.11	98.62	155.41	208.62
5′	1.35	2.460	3.21	14.40	26.88	1.305	18.79	35.08	270.6	943.0	505.2	47.32	63.54	89.30	110.00
6′	1.37	2.571	3.20	17.60	26.94	1.245	21.91	33.54	385.6	903.6	590.3	25.41	30.00	41.98	46.46
7′	1.67	4.657	3.22	20.80	26.83	.691	14.37	18.54	298.9	497.4	385.6	11.04	11.46	16.57	16.46
8′	2.25	11.391	3.24	24.00	26.62	.284	6.82	7.56	163.7	201.2	181.4	9.22	3.90	5.53	5.00
9′	3.12	30.371	3.26	27.20	26.18	.107	2.91	2.80	79.2	73.3	76.2	1.31	1.10	1 31	1.10
10′	4 25	76.765	3.29	30.40	25.60	.043	1.31	1.10	39.8	28.2	33.4	0	0	0	0
								Σ =	1,439	4,656	2,329				

1	17	18	19	20	21	22	23	24	25	26	27
Point or a	$\frac{\sum \text{col. } 12}{1{,}000}$ ×col. 15	$\frac{\sum \text{col. } 10}{1{,}000}$ ×col. 16	$\frac{\sum \text{col. } 11}{1{,}000}$ ×col. 15	$\frac{\sum \text{col. } 12}{1{,}000}$ ×col. 16	(col. 17−col. 18) ×dx	(col. 19−col. 20) ×dx	H= col. 21 ÷C	V= col. 22 ÷C	(col. 24 ×l)−(col. 23 ×h)	l−col. 5	$M_{B'}$= col. 26−col. 25
0_1	0	0	0	0	0	0	0	0	0	0	0
0_2	0	0	0	0	0	0	0	0	0	0	0
0_3	0	0	0	0	0	0	0	0	0	0	0
0_4	0	0	0	0	0	0	0	0	0	0	0
1′	945.2	928.4	1,889.8	1,502 5	53.8	1,639.3	.042	.971	30.01	30.40	+.39
2′	742.8	698.8	1,485.1	1,132.8	140.8	1,132.8	.110	.888	25.64	27.20	+1.56
3′	545.2	485.2	1,090.1	785.3	192.0	975.4	.150	.764	20.65	24.00	+3.35
4′	361.9	300.1	723.5	485.8	197.8	760.6	.155	.596	15.15	20.80	+5.65
5′	207.9	158.3	415.7	256.1	158.7	510.7	.124	.400	9.66	17.60	+7.94
6′	97.8	66.8	195.4	108.2	99.2	279.0	.078	.219	5.04	14.40	+9.36
7′	38.6	23.7	77.1	38.3	47.7	124.2	.037	.097	2.16	11.20	+9.04
8′	12.9	7.2	25.7	11.6	18.2	45.1	.014	.035	.77	8.00	+7.23
9′	3.1	1.6	6.1	2.6	4.8	11.2	.004	.009	.19	4.80	+4.61
10′	0	0	0	0	0	0	0	0	0	1.60	+1.60

h (arch rib) = 25.3 ft. l (arch rib) = 32.0 ft. dx = 3.20 ft.

$$C=\frac{\sum \text{col. } 10 \sum \text{col. } 11-(\sum \text{col. } 12)^2}{1{,}000}=1{,}276$$

$$\text{Moment stiffness of arch rib}=\frac{l\left(\frac{l\sum \text{col. } 11}{1{,}000}-\frac{h\sum \text{col. } 12}{1{,}000}\right)+h\left(\frac{h\sum \text{col } 10}{1{,}000}-\frac{l\sum \text{col. } 12}{1{,}000}\right)}{12\ C}=0.125$$

$$\text{Induced thrust of arch rib}=\left(\frac{l\sum \text{col. } 12}{1{,}000}-\frac{h\sum \text{col. } 10}{1{,}000}\right)\div(\text{moment stiffness}\times 12\ C)=0.020$$

$$\text{Thrust stiffness of arch rib}=\frac{\sum \text{col. } 10}{1{,}000}\div 12\ C=0.00009$$

$$\text{Induced moment of arch rib}=\left(l\,\frac{\sum \text{col. } 12}{1{,}000}-h\,\frac{\sum \text{col. } 10}{1{,}000}\right)\div\frac{\sum \text{col. } 10}{1{,}000}=26.5$$

$$H_T=\frac{ETle\times\frac{\sum \text{col. } 10}{1{,}000}}{12\ C} \qquad V_T=\frac{ETle\times\frac{\sum \text{col. } 12}{1{,}000}}{12\ C}$$

Table 2.—*Pier constants*

l=22.5 ft. t=2.0 ft.

$I=t^3/12=0.667$

Expression	Value
Moment stiffness = $3I\div l$ =	0.089
Induced thrust = $1\div l$ =	.044
Thrust stiffness = $3I\div l^3$ =	.00018
Induced moment = l =	22.5

Table 3.—*Distribution of moment and thrust stiffness*

Member	Moment stiffness		Thrust stiffness	
	Value	Distribution factor	Value	Distribution factor
		Percent		*Percent*
Arch rib, span 1	0.130	38	0.00020	43
Arch rib, span 2	.125	36	.00009	19
Pier	.089	26	.00018	38
Total	0.344	100	0.00047	100

Table 4.—*Distribution of induced moments and thrusts*

Member	Induced moment	Induced thrust
Arch rib, span 1	19.1 × distributed thrusts	0.030 × distributed moments.
Arch rib, span 2	26.5 × distributed thrusts	0.020 × distributed moments.
Pier	22.5 × distributed thrusts	0.044 × distributed moments.

The structure (fig. 8) used for the sample analysis is unsymmetrical both horizontally and vertically. It has been chosen to emphasize the important advantage possessed by this method, in common with ordinary moment distribution, of being applicable to unsymmetrical as well as symmetrical structures with almost equal facility. In structures of this type the three frame footings are frequently at different elevations due to differences in the elevation of satisfactory foundation material. Horizontal dissymmetry has been less common in the past, but may be expected to occur more frequently in the construction of modern highways and interchanges, requiring numerous structures which must fit the alinements and clearances otherwise determined.

The sequence of the various steps in the sample analysis follows that used in the development of the method. The procedure as applied to an actual analysis is as follows:

1. A working drawing of the frames is made, and required basic data are scaled and entered in the tabulation forms.
2. The tabulation forms are then completed by the calculations indicated in the column headings, and the expressions below the tables are computed.
3. Moment and thrust distribution factors are computed.
4. A unit moment and a unit thrust are distributed individually at each side of the joint (at one side only if the structure is symmetrical).
5. Final distributed moments and thrusts are obtained by simple proportion to the unit distributed moments and thrusts.

1.—Construction and measurement of working drawing

The structural frames are laid out to a convenient scale, as shown in figure 8. Sufficient accuracy usually may be obtained with a scale of one-fourth or one-half inch equals one foot. The neutral axis lines of the arch ribs are drawn midway between the face surfaces. The neutral axis lines of the frame legs are drawn as perpendicular lines bisecting the bases of the legs at the footing tops. This involves a slight inaccuracy, but eliminates unwarranted refinement. The design spans between the neutral axis lines of the piers and frame legs are each divided into 10 equal horizontal parts which are projected vertically onto the neutral axis lines of the arch ribs, making 10 *ds* divisions.

The centers of gravity of the *ds* divisions are for convenience assumed to be at their midpoints in horizontal projection. These centers of gravity are numbered 1 through 10 for span 1 (the left arch) and 1′ through 10′ for span 2 (the right arch). Four longitudinal divisions are made of the frame leg neutral axis lines, and the midpoints of these *ds* divisions are located and designated O_1 through O_4. The lengths of the *ds* divisions are scaled and are entered in column 4 of the tabulation forms, tables 1 (a) and 1 (b), opposite the proper points. In the analysis, a load of unity is placed successively at each numbered point on the arch ribs.

Values of t, x, and y at each load point are obtained by scaling, and are recorded in columns 2, 5, and 6 on the tabulation forms. The required x and y values are, respectively, the horizontal distance from the neutral axis line of the frame leg and the vertical distances from a level line through the hinge, to the numbered load points. The t values are the thicknesses of the various sections, measured radially through each load point. The remaining data for the analysis are derived in the tabulation from these basic measurements.

2.—Completion of tabulation forms

Moments of inertia of the sections are computed in column 3 as t^3 instead of the true value, $t^3 \div 12$, in order to avoid large figures in the subsequent columns. It is necessary, however, to reinsert the factor 12 in some of the final expressions in order to make them applicable to a section 1 foot wide instead of 12 feet wide, and this is done in the stiffness and thrust expressions and also in the expressions for H_t and V_t, which appear below the tabulations in tables 1 (a) and 1 (b).

Entries for columns 7 to 12, inclusive, are computed as indicated by the column heads. Note that totals are recorded for columns 10, 11, and 12.

The method of computing entries for columns 13 to 16, inclusive, involves the summation process, and an explanation of this procedure may be helpful. Assume that it is desired to solve for H and M_B due to a load of unity at point 1. Point 1 is therefore taken as a. The entry for column 13 is to be the summation from $a+1$ to B of $x\Delta$, which is the sum of the $x\Delta$ values in column 8 from point 2 to point 10, inclusive. The entry for column 14 is similarly computed except that $y\Delta$ values, in column 9, are used for summation.

The entry in column 15 is to be the summation from a to B of column 13, and for point 1 this is the sum of the values in column 13 from point 1 to point 10, inclusive. The entry for column 16 is similarly computed except that values in column 14 are used for summation.

If H and M_B were desired only for a load of unity at point 1, it would simply be necessary to complete the operations indicated in columns 17 to 27, inclusive, along the line opposite point 1. In the sample analysis H and M_B are computed for 10 positions of the unit load on each arch, and the tabulations are completed in full.

In several of the columns of the tabulation forms, tables 1 (a) and 1 (b), and in the expressions below them, division by 1,000 is indicated. This is merely a device to avoid large figures, not a function of the basic formulas.

The constant C, which appears below the tabulation forms, is a term that is derived from the totals of columns 10, 11, and 12. It is evaluated independently for convenience in use in the computation of entries for columns 23 and 24 and for the expressions which appear below the form.

The computation of the pier constants is illustrated in table 2 (bottom of page 71).

3.—Computation of moment and thrust distribution factors

The values derived in the expressions below the tabulation forms in tables 1 (a) and 1 (b), and in table 2, are used in obtaining relative values of moment and thrust stiffness, and values of induced moments and thrusts.

The moment and thrust distribution factors are evaluated in exactly the same manner as in ordinary moment distribution. This is illustrated in table 3. The moment stiffness values for the structure members, derived in tables 1 (a), 1 (b), and 2, are entered in the proper column and are totaled. Each value is divided by the total, yielding the distribution factor. The thrust stiffness distribution is handled in the same manner.

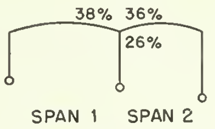
The method of distributing the induced moments and thrusts is shown in table 4, for which the figures were derived in tables 1 (a), 1 (b), and 2. The values are constants by which distributed moments and distributed thrusts are multiplied, for purposes described in the next step of the procedure.

4.—Distribution of a unit moment and a unit thrust

Completion of the tabulation forms, tables 1 (a) and 1 (b), provides the fixed-end moments and their correlated fixed-end thrusts due to a unit load at 10 positions on each arch. The juncture of the arch ribs and pier has thus far been considered completely fixed, so that transfer of moment or thrust from one member to the others is not permitted. The next step consists of distributing each correlated fixed-end moment and fixed-end thrust so that for each position of the unit load actual moments at the joint and reactions at all three footings are obtained.

Tables 5–8.—*Unit moment and thrust distributions*

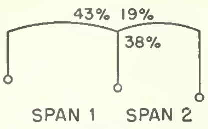
MOMENT DISTRIBUTION FACTORS



INDUCED THRUSTS

0.030 × moment (arch rib, span 1)
0.020 × moment (arch rib, span 2)
0.044 × moment (pier)

THRUST DISTRIBUTION FACTORS



INDUCED MOMENTS

19.1 × thrust (arch rib, span 1)
26.5 × thrust (arch rib, span 2)
22.5 × thrust (pier)

Table 5.—*Distribution of unit moment, span 1*

	Rib	Pier	Rib		Rib	Pier	Rib	
Fixed-end moment	−1.000	0	0		0	0	0	Fixed-end thrust
Balancing moment	+.380	+.260	+.360	Induced thrust→	+.011	−.011	+.007	
	−.057	+.068	−.027	←Induced moment	−.003	−.003	−.001	Balancing thrust
Balancing moment	+.006	+.004	+.006	Induced thrust→	----	----	----	
	----	-----	----	←Induced moment	----	---	---	Balancing thrust
Balancing moment	----	-----	----		+.008	−.014	+.006	Final thrusts
Final moments	−.671	+.332	+.339					

Table 6.—*Distribution of unit thrust, span 1*

	Rib	Pier	Rib		Rib	Pier	Rib	
Fixed-end moment	0	0	0		+1.000	0	0	Fixed-end thrust
	−8.213	+8.550	−5.035	←Induced moment	−.430	−.380	−.190	Balancing thrust
Balancing moment	+1.785	+1.221	+1.691	Induced thrust→	+.054	−.054	+.034	
	−.287	+.293	−.159	←Induced moment	−.015	−.013	−.006	Balancing thrust
Balancing moment	+.058	+.040	+.055	Induced thrust→	+.002	−.002	+.001	
Final moments	−6.657	+10.104	−3.448		-------	------	------	Balancing thrust
					+.611	−.449	−.161	Final thrusts

Table 7.—*Distribution of unit moment, span 2*

	Rib	Pier	Rib		Rib	Pier	Rib	
Fixed-end moment	0	0	+1.000		0	0	0	Fixed-end thrust
Balancing moment	−.380	−.260	−.360	Induced thrust→	−.011	+.011	−.007	
	+.057	−.068	+.027	←Induced moment	+.003	+.003	+.001	Balancing thrust
Balancing moment	−.006	−.004	−.006	Induced thrust→	-------	-------	-------	
	------	---------	-------	←Induced moment	-------	-------	-------	Balancing thrust
Balancing moment	-------	---------	-------		−.008	+.014	−.006	Final thrusts
Final moments	−.329	−.332	+.661					

Table 8.—*Distribution of unit thrust, span 2*

	Rib	Pier	Rib		Rib	Pier	Rib	
Fixed-end moment	0	0	0		0	0	−1.000	Fixed-end thrust
	+8.213	−8.550	+5.035	←Induced moment	+.430	+.380	+.190	Balancing thrust
Balancing moment	−1.785	−1.221	−1.691	Induced thrust→	−.054	+.054	−.034	
	+.287	−.293	+.159	←Induced moment	+.015	+.013	+.006	Balancing thrust
Balancing moment	−.058	−.040	−.055	Induced thrust→	−.002	+.002	−.001	
	------	-------	------		------	------	------	Balancing thrust
Final moments	+6.657	−10.104	+3.448		+.389	+.449	−.839	Final thrusts

Since 20 positions of the unit load on both arches are considered, it would appear that a prohibitive number of distributions is required. Actually, only two distributions are required for symmetrical structures, and four if the structure is unsymmetrical.

Referring to table 5 in the sample analysis, it will be noted that a fixed-end moment of unity is applied to the joint at the juncture of the left arch rib. This moment is distributed in a manner similar to that of ordinary moment distribution, using the moment distribution factors computed in table 3 and shown around the joint in the left-hand sketch above table 5. The values of the distributed moments are shown in the left tabulation of table 5. This shows, first, the unit moment of −1.000 opposite the stub "fixed-end moment," and, immediately below, the distributing moments opposite the stub "balancing moments."

Next, the first arrow, "induced thrusts," is followed to the tabulation at the right, and the thrusts induced by the balancing moments are entered. These values are obtained by multiplying each balancing moment by the corresponding induced thrust constants, previously computed in table 4 and repeated for convenience under the sketch above table 5.

Next, these unbalanced thrusts are balanced exactly as if they were unbalanced moments, but using the thrust distribution factors computed in table 3 and shown around the joint in the right-hand sketch above table 5. Now the second arrow, "induced moments," is followed to the left, and the new moments induced by the balancing thrusts are entered. These values are obtained by multiplying each balancing thrust by the corresponding induced moment constants, previously computed in table 4 and repeated for convenience under the sketch above table 5.

This procedure is repeated until the converging values become so small that further refinement is unnecessary. As shown in the actual example, convergence occurs rapidly.

In table 6 a fixed-end thrust of unity is distributed in the same manner as described above for a unit moment, and final values of moments and thrusts are obtained.

The significance of these procedures may be summarized as follows:

(*a*) A load is placed on one of the frames but the joint at the middle is fixed, so that the moment at the joint and the outward "kick," or thrust, are so-called "fixed-end" values.

(*b*) Considering the fixed-end moment and fixed-end thrust independently of each other, the fixity at the joint is first released, and then final values of moment and thrust due only to the fixed-end moment are computed. Computation is similarly made of final values of moment and thrust due only to the fixed-end thrust.

(*c*) Individual values are now added algebraically to obtain final values due to the load that caused the fixed-end moment and fixed-end thrust.

The structure analyzed in the example is unsymmetrical, and the process is therefore repeated for the right arch in tables 7 and 8 It is important to note, however, that all the distributed values in the distribution for the right arch are identical with those for the left arch, with signs changed. This fact saves considerable time in the second procedure.

5.—Evaluation of final distributed moments and thrusts

The manner of obtaining final distributed moments and thrusts for span 1 is illustrated in tables 9 (a), 9 (b), and 9 (c) of the sample analysis. In table 9 (a) are recorded the values of M_B, $M_{B'}$, H_1, H_2, and H_3 due to a distributed fixed-end moment of unity; and similar values due to a distributed fixed-end thrust of unity are recorded in table 9 (b). These values are, of course, obtained from tables 5 and 6.

In the second column of table 9 (c) the fixed-end moments at each load point are recorded as obtained in column 27 of table 1 (a). In the third column the fixed-end thrusts at each load point are recorded as obtained in column 23 of table 1 (a).

Each of the values in table 9 (a) is then multiplied by the actual fixed-end moment for each load point and the results recorded. Similarly, each of the values in table 9 (b) is multiplied by the actual fixed-end thrust for each load point. The sums of the corresponding pairs of values obtained by these two series of multiplications are then recorded, and are the final distributed values of moment and thrust for a load of unity at each load point.

The final distributed moments and thrusts for span 2 are obtained in the same manner, as illustrated in tables 10 (a), 10 (b), and 10 (c) of the sample analysis.

Table 9.—*Tabulation of final moments and thrusts, span 1*

Table 9 (a)

Fixed-end moment=−1.000				
M_B	$M_{B'}$	H_1	H_2	H_3
−0.671	+0.339	+0.008	−0.014	+0.006

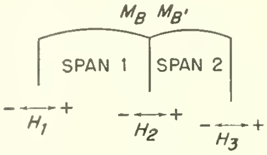


Table 9 (b)

Fixed-end thrust=+1.000				
M_B	$M_{B'}$	H_1	H_2	H_3
−6.657	−3.448	+0.611	−0.449	−0.161

Table 9 (c)

Unit load at point—	Fixed-end moment	Fixed-end thrust	M and H due to fixed-end moment					M and H due to fixed-end thrust					Final values of M and H				
			M_B	$M_{B'}$	H_1	H_2	H_3	M_B	$M_{B'}$	H_1	H_2	H_3	M_B	$M_{B'}$	H_1	H_2	H_3
1	−0.20	+0.094	−0.13	+0.07	+0.002	−0.003	+0.001	−0.62	−0.32	+0.058	−0.042	−0.015	−0.75	−0.25	+0.060	−0.045	−0.014
2	−1.11	+.254	−.72	+.38	+.010	−.017	+.007	−1.68	−.87	+.156	−.115	−.041	−2.40	−.49	+.166	−.132	−.037
3	−2.86	+.365	−1.86	+.97	+.026	−.043	+.017	−2.41	−1.25	+.224	−.165	−.058	−4.27	−.28	+.250	−.208	−.041
4	−5.53	+.400	−3.59	+1.88	+.050	−.083	+.033	−2.65	−1.37	+.245	−.181	−.064	−6.24	+.51	+.295	−.264	−.031
5	−8.81	+.345	−5.73	+3.00	+.079	−.132	+.053	−2.28	−1.19	+.211	−.156	−.055	−8.01	+1.81	+.290	−.288	−.002
6	−11.44	+.236	−7.44	+3.89	+.102	−.172	+.069	−1.56	−.81	+.145	−.107	−.038	−9.00	+3.08	+.247	−.279	+.031
7	−12.03	+.126	−7.82	+4.09	+.108	−.180	+.072	−.83	−.43	+.077	−.057	−.020	−8.65	+3.66	+.185	−.237	+.052
8	−10.23	+.051	−6.65	+3.48	+.092	−.153	+.061	−.34	−.18	+.031	−.023	−.008	−6.99	+3.30	+.123	−.176	+.053
9	−6.77	+.013	−4.40	+2.30	+.061	−.102	+.041	−.09	−.04	+.008	−.006	−.002	−4.49	+2.26	+.069	−.108	+.039
10	−2.41	0	−1.57	+.82	+.022	−.036	+.014	0	0	0	0	0	−1.57	+.82	+.022	−.036	+.014

Table 10.—*Tabulation of final moments and thrusts, span 2*

Table 10 (a)

Fixed-end moment=+1.000				
M_B	$M_{B'}$	H_1	H_2	H_3
−0.329	+0.661	−0.008	+0.014	−0.006

Table 10 (b)

Fixed-end thrust=−1.000				
M_B	$M_{B'}$	H_1	H_2	H_3
+6.657	+3.448	+0.389	+0.449	−0.839

Table 10 (c)

Unit load at point—	Fixed-end moment	Fixed-end thrust	M and H due to fixed-end moment					M and H due to fixed-end thrust					Final values of M and H				
			M_B	$M_{B'}$	H_1	H_2	H_3	M_B	$M_{B'}$	H_1	H_2	H_3	M_B	$M_{B'}$	H_1	H_2	H_3
10′	+1.60	0	−0.56	+1.06	−0.014	+0.024	−0.010	0	0	0	0	0	−0.56	+1.06	−0.014	+0.024	−0.010
9′	+4.61	−.004	−1.61	+3.04	−.041	+.069	−.028	+.026	+.014	+.002	+.002	−.003	−1.58	+3.05	−.039	+.071	−.031
8′	+7.23	−.014	−2.53	+4.77	−.065	+.108	−.043	+.093	+.048	+.005	+.006	−.012	−2.44	+4.82	−.060	+.114	−.055
7′	+9.04	−.037	−3.16	+5.97	−.081	+.136	−.054	+.245	+.127	+.014	+.017	−.031	−2.91	+6.10	−.067	+.153	−.085
6′	+9.36	−.078	−3.28	+6.18	−.084	+.140	−.056	+.516	+.268	+.030	+.035	−.066	−2.76	+6.45	−.054	+.175	−.122
5′	+7.94	−.124	−2.78	+5.24	−.071	+.119	−.048	+.820	+.426	+.048	+.056	−.104	−1.96	+5.67	−.023	+.175	−.152
4′	+5.65	−.155	−1.98	+3.73	−.051	+.085	−.034	+1.025	+.533	+.060	+.070	−.130	−0.95	+4.26	+.009	+.155	−.164
3′	+3.35	−.150	−1.17	+2.21	−.030	+.050	−.020	+.992	+.515	+.058	+.068	−.126	−0.18	+2.73	+.028	+.118	+.146
2′	+1.56	−.110	−.55	+1.03	−.014	+.023	−.009	+.728	+.378	+.043	+.050	−.092	+0.18	+1.41	+.029	+.073	−.101
1′	+.39	−.042	−.14	+.26	−.004	+.006	−.002	+.278	+.144	+.016	+.019	−.035	+0.14	+.40	+.012	+.025	−.037

Required Design Constants

The joint constants for the condition of fixed footings include all of the design constants used in the analysis for hinged footings with identical definitions (see page 66). To this group are added the following:

Induced moment at footing.—The moment induced at the footing by a horizontal thrust at B without vertical displacement or rotation at B.

Moment carry-over.—The ratio of moment induced at the footing to an applied moment at B without horizontal or vertical displacement at B.

Development of Fixed-End Moment and Fixed-End Thrust

From the conditions that point A is not displaced horizontally, that point A is not displaced vertically, and that no rotation of the end tangents occurs, as shown in figure 9, the following elastic equations may be written:

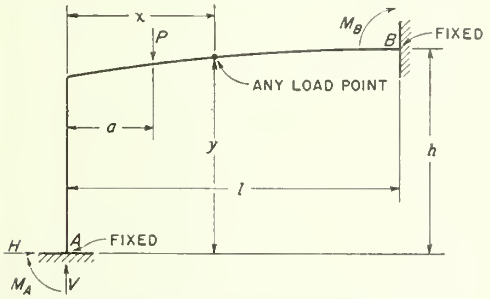


Figure 9.—Sketch for use in deriving fixed-end moments and thrusts.

$$\sum_A^B M\frac{ds}{EI}\ \text{(change in angle between end tangents)}=0 \text{------------- } (35)$$

$$\sum_A^B Mx\frac{ds}{EI}\ \text{(vertical displacement of } A)=0 \qquad (36)$$

$$\sum_A^B My\frac{ds}{EI}\ \text{(horizontal displacement of } A)=0 \qquad (37)$$

The moment at any point between A and $a = M_A + Vx - Hy$.

The moment at any point between a and B, when P is unity, $= M_A + Vx - Hy - (x-a)$.

For convenience, the following symbols are adopted:

$$\frac{ds}{I}=\Delta \qquad \sum_A^B x\Delta = B \qquad \sum_A^B x^2\Delta = D$$

$$\sum_A^B \Delta = A \qquad \sum_A^B y\Delta = C \qquad \sum_A^B y^2\Delta = E$$

$$\sum_A^B xy\Delta = F \qquad \sum_a^B (x-a)x\Delta = K_2$$

$$\sum_a^B (x-a)\Delta = K_1 \qquad \sum_a^B (x-a)y\Delta = K_3$$

E, being constant for the entire structure, may be eliminated from the basic equations in evaluating external moments and reactions due to flexure.

Inserting the general expression for moment in equations (*35*), (*36*), and (*37*), and using the abbreviated notation, the following equations are obtained:

$$M_AA+VB-HC-K_1=0 \text{--------- } (38)$$

$$M_AB+VD-HF-K_2=0 \text{--------- } (39)$$

$$M_AC+VF-HE-K_3=0 \text{--------- } (40)$$

Solving equations (*38*) and (*39*):

$$V\left(\frac{B^2}{A}-D\right)-H\left(\frac{CB}{A}-F\right)-K_1\frac{B}{A}+K_2=0 \qquad (41)$$

Solving equations (*39*) and (*40*):

$$V\left(\frac{DC}{B}-F\right)-H\left(\frac{FC}{B}-E\right)-K_2\frac{C}{B}+K_3=0 \qquad (42)$$

Solving equations (*41*) and (*42*) for H, and clearing:

$$H=\frac{K_3\left(\frac{B^2}{A}-D\right)-K_2\left[\frac{C}{B}\left(\frac{B^2}{A}-D\right)+\left(\frac{DC}{B}-F\right)\right]+K_1\frac{B}{A}\left(\frac{DC}{B}-F\right)}{\left(\frac{B^2}{A}-D\right)\left(\frac{FC}{B}-E\right)-\left(\frac{CB}{A}-F\right)\left(\frac{DC}{B}-F\right)} \text{--------- } (43)$$

For further simplification the following symbols are used:

$$C_1=\frac{C}{B} \qquad C_6=\frac{CB}{A}-F$$

$$C_2=\frac{B}{A} \qquad C_7=\frac{D}{B}$$

$$C_3=\frac{B^2}{A}-D \qquad C_8=\frac{AD}{B}-B$$

$$C_4=\frac{DC}{B}-F \qquad C_H=C_3C_5-C_6C_4$$

$$C_5=\frac{FC}{B}-E$$

Substituting these symbols in equation (*43*):

$$H=\frac{K_1C_2C_4-K_2C_6+K_3C_3}{C_H} \text{------ } (44)$$

Substituting the symbols in equation (*41*):

$$V=\frac{HC_6+K_1C_2-K_2}{C_3} \text{--------- } (45)$$

From equations (*38*) and (*39*):

$$M_A=\frac{HC_4+K_1C_7-K_2}{C_8} \text{--------- } (46)$$

The expressions for M_A, H, and V are evaluated by means of the tabular computation form, illustrated in the sample analysis. As in the tabular form for hinged footings, the expressions

$$\sum_a^B (x-a)\Delta,\ \sum_A^B (x-a)x\Delta,\ \text{and}\ \sum_a^B (x-a)y\Delta$$

are equated to the expressions

$$\sum_a^B \sum_{a+1}^B \Delta dx,\ \sum_a^B \sum_{a+1}^B x\Delta dx,\ \text{and}\ \sum_a^B \sum_{a+1}^B y\Delta dx$$

The general procedure in adapting the various expressions to a tabular computation form is also similar to that employed in the analysis for hinged footings, and reference thereto will be helpful in studying the form as arranged for fixed footings.

Development of Arch Rib Design Constants

The arch rib design constants are derived in two steps. First the rib is given a unit rotation at B, and H_1, V_1, M_{1A}, and M_{1B} are computed. No horizontal or vertical displacement is permitted in this step. Then the rib is given a unit horizontal displacement without vertical displacement, and H_2, V_2, M_{2A}, and M_{2B} are computed. These values of M, V, and H are then combined as required to obtain all the necessary joint constants, as well as expressions for H_T and V_T caused by temperature change.

In deriving the expressions, the system of notation used in obtaining expressions for fixed-end moments and thrusts is adopted. Values obtained in the computation for fixed-end moments and thrusts are used in evaluating the expressions for the joint constants. No additional basic computation is required.

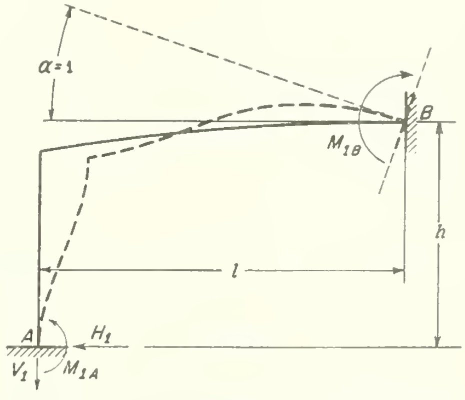


Figure 10.—Sketch for use in step 1 derivations.

Step 1.—The arch rib is given a unit rotation at B and no horizontal or vertical displacement at A is permitted, as shown in figure 10. Then:

$$M_{1A}A+V_1B-H_1C=1 \text{--------- } (47)$$

$$M_{1A}B+V_1D-H_1F=l \text{--------- } (48)$$

$$M_{1A}C+V_1F-H_1E=h \text{--------- } (49)$$

Solving equations (*47*) and (*48*):

$$V_1\left(\frac{B^2}{A}-D\right)-H_1\left(\frac{CB}{A}-F\right)=\frac{B}{A}-l \text{ - - } (50)$$

Solving equations (*48*) and (*49*):

$$V_1\left(\frac{DC}{B}-F\right)-H_1\left(\frac{FC}{B}-E\right)=l\frac{C}{B}-h \text{ - - } (51)$$

The coefficients 10, 100, and 10,000 are introduced in the following equations to make the expressions for H_1 V_1, and M_{1A} applicable to the computation form used in the sample analysis, in which actual values of x and y are divided by 10, reducing the numerical order of the derived quantities.

Solving equations (*50*) and (*51*), and clearing:

$$H_1=\frac{(lC_1-h)100C_3-(10C_2-l)100C_4}{-10{,}000C_H} \text{ - - - } (52)$$

From equation (*50*):

$$V_1=\frac{H_1(100C_6)+10C_2-l}{100C_3} \text{ - - - - - - } (53)$$

From equations (*47*) and (*48*):

$$M_{1A}=\frac{H_1(100\ C_4)+10\ C_7-l}{10\ C_8} \text{ - - - - - } (54)$$

From statics:

$$M_{1B}=M_{1A}+V_1l-H_1h \text{ - - - - - - } (55)$$

Step 2.—The arch rib is given a unit horizontal displacement, without vertical displacement, equal to the arch height, h, as shown in figure 11. Then:

$$M_{2A}C+VF-HE=h \text{ - - - - - - - } (56)$$

$$M_{2A}B+VD-HF=0 \text{ - - - - - - - } (57)$$

$$M_{2A}A+VB-HC=0 \text{ - - - - - - - } (58)$$

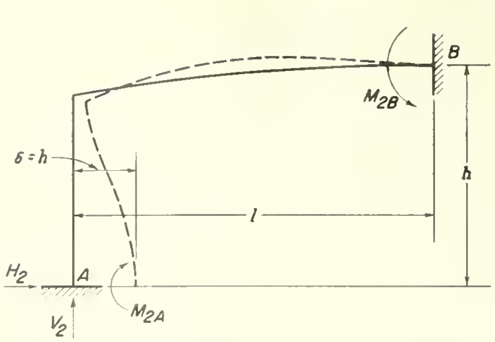


Figure 11.—Sketch for use in step 2 derivations.

Solving equations (*56*) and (*57*):

$$V\left(\frac{DC}{B}-F\right)-H\left(\frac{FC}{B}-E\right)=-h \text{ - - - } (59)$$

Solving equations (*57*) and (*58*):

$$V\left(\frac{B^2}{A}-D\right)-H\left(\frac{CB}{A}-F\right)=0 \text{ - - - - } (60)$$

Solving equations (*59*) and (*60*), and clearing:

$$H_2=\frac{-h(100C_3)}{-1{,}000C_H} \text{ - - - - - - - - - - } (61)$$

From equation (*60*):

$$V_2=\frac{H_2C_6}{C_3} \text{ - - - - - - - - - - - - - } (62)$$

Equations (*61*) and (*62*) may be used in evaluating H_T and V_T due to temperature change by substituting ± $(ETle)$ ÷ 12 for h. The factor 12 is inserted to correct for the use of t^3 instead of $t^3/12$ (where $t=$ radial depths of sections at load points) for values of I in the computation form.

Solving equations (*57*) and (*58*) for M_{2A}:

$$M_{2A}=\frac{H_2(100C_4)}{10C_8} \text{ - - - - - - - - - - } (63)$$

From statics:

$$M_{2B}=M_{2A}+V_2l-H_2h \text{ - - - - - - - - - } (64)$$

Using the values of H_1, V_1, M_{1A}, M_{2A}, H_2, V_2, M_{2A}, and M_{2B}, and inserting the factor 12 where required to correct for the use of t^3 instead of $t^3/12$ in the computation form, the joint constants for the arch ribs may be expressed as follows:

Moment stiffness $=M_{1B}\div 12$

Induced thrust $=H_1\div M_{1B}$

Thrust stiffness $=H_2\div 12\ h$

Induced moment (at pier top) $=M_{2B}\div H_2$

Induced moment (at footing) $=-M_{2A}\div H_2$

Moment carry-over $=-M_{1A}\div M_{1B}$

Development of Elastic Pier Constants

The elastic pier constants are derived (see fig. 12) by giving A a unit rotation without vertical displacement, and evaluating M_A, the moment stiffness, and P/M, the induced thrust. P is the force developed in restraining the ends against displacement. Moment carry-over is expressed by the term, M_B/M_A.

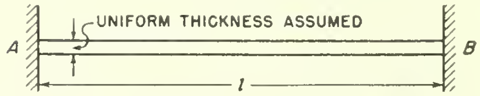


Figure 12.—Sketch for use in deriving elastic pier constants.

Point A is then given a vertical displacement, $\delta=1$, from which the thrust stiffness, P/δ, and induced moment, M_A/P, are evaluated.

Performing these operations, the following expressions for the elastic pier constants are obtained:

Moment stiffness $=4\ I\div l$ (relative)
Induced thrust $=3\div 2l$
Thrust stiffness $=12\ I\div l^3$ (relative)
Induced moment $=l\div 2$
Moment carry-over $=+0.5$

SAMPLE ANALYSIS—II

Two-span arched frame with elastic pier, unsymmetrical both horizontally and vertically, and with fixed footings.

Application of Method

Evaluation of the design constants for arch ribs and pier of a two-span arched frame with elastic pier, unsymmetrical both horizontally and vertically, and with fixed footings, is illustrated in the sample analysis that follows. The structure is identical with that analyzed in part I (fig. 8), except that the footings are fixed instead of hinged.

The general procedure and sign convention are closely similar to those used in the analysis of the structure with hinged footings. A working drawing of the frames is made to convenient scale, from which values of t, ds, x, and y are scaled. These values are entered in tables 11 (a) and 11 (b), the scaled x and y dimensions first being divided by 10.

The tables are then completed by the computations indicated in the column headings. It will be noted that columns 7 to 12, inclusive, are each totaled to obtain values of A, B, C, D, E, and F, which in turn are used to obtain the C "subscript" series of values. The latter are used in computation of entries in some of the table columns.

The C "subscript" values are also used in computing the moments, vertical reactions, and thrusts for each span, as shown in table 11 (c). These in turn are used to derive arch rib joint constants as illustrated in table 11 (d). Derivation of the pier constants appears in table 12.

Moment and thrust stiffness distribution factors are computed in table 13, and the method of distributing induced moments and thrusts and moment carry-overs is shown in table 14. Tables 15, 16, 17, and 18 illustrate the distribution of a unit moment and a unit thrust for each of the two spans. The evaluation of final distributed moments and thrusts is shown in tables 19 and 20.

Additional computations are provided in tables 21 and 22 for obtaining moments induced at the footing by balancing thrusts, and moments carried over by balancing moments.

Table 11 (a).—*Fixed-end moments and fixed-end thrusts, span l*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Point or *a*	*t*	(col. 2)^3	*ds*	$\frac{x}{10}$	$\frac{y}{10}$	col. 4 ÷ col. 3	col. 5 × col. 7	col. 6 × col. 7	col. 5 × col. 8	col. 6 × col. 9	col. 5 × col. 9	$\sum_{a+1}^{B}$ col. 7	$\sum_{a+1}^{B}$ col. 8	$\sum_{a+1}^{B}$ col. 9	$\sum_{a}^{B}$ col. 13 ×0.1*dx*	$\sum_{a}^{B}$ col. 14 ×0.1*dx*	$\sum_{a}^{B}$ col. 15 ×0.1*dx*
0_1	2.46	14.887	4.87	0	0.243	0.327	0	0.080	0	0.019	0	0	0	0	0	0	0
0^2	2.78	21.485	4.87	0	.730	.227	0	.166	0	.121	0	0	0	0	0	0	0
0_3	3.17	31.855	4.87	0	1.217	.153	0	.186	0	.226	0	0	0	0	0	0	0
0_4	3.45	41.064	4.87	0	1.704	.119	0	.203	0	.346	0	0	0	0	0	0	0
1	3.33	36.926	4.92	.241	1.992	.133	.032	.265	.008	.528	.064	5.585	13.042	11.960	11.697	31.330	25.088
2	2.58	17.174	4.88	.723	2.058	.284	.205	.584	.148	1.202	.422	5.301	12.837	11.376	9.005	25.044	19.323
3	2.04	8.490	4.85	1.205	2.108	.571	.688	1.204	.829	2.538	1.451	4.730	12.149	10.172	6.450	18.857	13.840
4	1.70	4.913	4.81	1.687	2.142	.979	1.652	2.097	2.787	4.492	3.538	3.751	10.497	8.075	4.170	13.001	8.937
5	1.60	4.096	4.81	2.169	2.160	1.174	2.546	2.536	5.522	5.478	5.501	2.577	7.951	5.539	2.362	7.941	5.045
6	1.60	4.096	4.81	2.651	2.164	1.174	3.112	2.541	8.250	5.499	6.736	1.403	4.839	2.998	1.120	4.109	2.375
7	1.83	6.128	4.81	3.133	2.158	.785	2.459	1.694	7.704	3.656	5.307	.618	2.380	1.304	.444	1.777	.930
8	2.33	12.649	4.85	3.615	2.133	.383	1.385	.817	5.007	1.743	2.953	.235	.995	.487	.146	.629	.301
9	3.08	29.218	4.88	4.097	2.092	.167	.684	.349	2.802	.730	1.430	.068	.311	.138	.033	.150	.067
10	4.17	72.512	4.92	4.579	2.035	.068	.311	.138	1.424	.281	.632	0	0	0	0	0	0
					$\Sigma=$	6.544 $=A$	13.174 $=B$	12.860 $=C$	34.481 $=D$	26.859 $=E$	28.034 $=F$						

1	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
Point or *a*	col. 16 × C_2C_4	col. 17 ×C_6	col. 18 ×C_3	col. 19 − col. 20	col. 21 + col. 22	$H=$ col. 23 $\div C_H$	col. 24 ×C_6	col. 16 ×C_2	col. 25 + col. 26 − col. 17	$V=$ col. 27 $\div C_3$	col. 24 ×C_4	col. 16 ×C_7	col. 29 + col. 30 − col. 17	$M_A=$ col. 31 ÷ $0.1C_8$	(col. 28 ×*l*)− (col. 29 ×*h*)	*l*− (10× col. 5)	$M_B=$ −col. 32 +col. 33 −col. 34
0_1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0_2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0_3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0_4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1	137.487	−73.344	−209.761	210.831	1.070	.137	−.321	+23.371	−8.280	.990	.806	30.845	+.321	−.77	+44.96	+45.79	−.06
2	105.845	−58.628	−161.560	164.473	2.913	.374	−.876	+17.992	−7.928	.948	2.200	23.746	+.902	−2.16	+38.18	+40.97	−.63
3	75.813	−44.144	−115.716	119.957	4.241	.545	−1.276	+12.887	−7.246	.867	3.206	17.009	+1.358	−3.24	+30.84	+36.15	−2.07
4	49.014	−30.435	−74.722	79.449	4.727	.607	−1.421	+8.332	−6.097	.728	3.571	10.996	+1.568	−3.75	+22.89	+31.33	−4.65
5	27.763	−18.590	−42.181	46.353	4.172	.536	−1.255	+4.719	−4.477	.535	3.153	6.229	+1.441	−3.44	+15.02	+26.51	−8.06
6	13.164	−9.619	−19.857	22.783	2.926	.376	−.880	+2.238	−2.751	.329	2.212	2.953	+1.056	−2.52	+8.30	+21.69	−10.87
7	5.219	−4.160	−7.776	9.379	1.603	.206	−.482	+.887	−1.372	.164	1.212	1.171	+.608	−1.45	+3.76	+16.87	−11.66
8	1.716	−1.472	−2.517	3.188	.671	.086	−.201	+.292	−.538	.064	.506	.385	+.262	−.63	+1.36	+12.05	−10.06
9	.388	−.351	−.560	.739	.179	.023	−.054	+.066	−.138	.017	.135	.087	+.072	−.17	+.36	+7.23	−6.70
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	+2.41	−2.41

$l=48.2$ ft. $h=20.1$ ft. $dx=4.82$ ft.

$$C_1=\frac{C}{B}=+0.984 \qquad C_6=\frac{CB}{A}-F=-2.341$$

$$C_2=\frac{B}{A}=+1.998 \qquad C_7=\frac{D}{B}=+2.637$$

$$C_3=\frac{B^2}{A}-D=-8.361 \qquad C_8=\frac{AD}{B}-B=+4.185$$

$$C_4=\frac{DC}{B}-F=+5.883 \qquad C_2C_4=+11.754$$

$$C_5=\frac{FC}{B}-E=+0.716 \qquad C_H=C_3C_5-C_6C_4=+7.786$$

Table 11 (b).—*Fixed-end moments and fixed-end thrusts, span 2*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Point or *a*	t	(col. 2)^3	ds	$\frac{x}{10}$	$\frac{y}{10}$	col. 4 ÷ col. 3	col. 5× col. 7	col. 6× col. 7	col. 5× col. 8	col. 6× col. 9	col. 5× col. 9	$\sum_{a+1}^{B}$ col. 7	$\sum_{a+1}^{B}$ col. 8	$\sum_{a+1}^{B}$ col. 9	$\sum_{a}^{B}$ col. 13 ×0.1dx	$\sum_{a}^{B}$ col. 14 ×0.1dx	$\sum_{a}^{B}$ col. 15 ×0.1dx
0_1	2.09	9.129	6.00	0	0.300	0.657	0	0.197	0	0.059	0	0	0	0	0	0	0
0_2	2.27	11.697	6.00	0	.900	.513	0	.462	0	.416	0	0	0	0	0	0	0
0_3	2.45	14.706	6.00	0	1.500	.408	0	.612	0	.918	0	0	0	0	0	0	0
0_4	2.63	18.191	6.00	0	2.100	.330	0	.693	0	1.455	0	0	0	0	0	0	0
1′	2.50	15.625	3.33	.160	2.454	.213	.034	.523	.005	1.283	.084	6.001	8.693	15.953	7.734	12.990	20.649
2′	1.96	7.530	3.27	.480	2.540	.434	.208	1.102	.100	2.799	.529	5.567	8.485	14.851	5.813	10.208	15.544
3′	1.62	4.252	3.25	.800	2.604	.764	.611	1.989	.489	5.179	1.591	4.803	7.874	12.862	4.032	7.493	10.792
4′	1.42	2.863	3.23	1.120	2.660	1.128	1.263	3.000	1.415	7.980	3.360	3.675	6.611	9.862	2.495	4.973	6.676
5′	1.35	2.460	3.21	1.440	2.688	1.305	1.879	3.508	2.706	9.430	5.052	2.370	4.732	6.354	1.319	2.858	3.520
6′	1.37	2.571	3.20	1.760	2.694	1.245	2.191	3.354	3.856	9.036	5.903	1.125	2.541	3.000	.561	1.343	1.487
7′	1.67	4.657	3.22	2.080	2.683	.691	1.437	1.854	2.989	4.974	3.856	.434	1.104	1.146	.201	.530	.527
8′	2.25	11.391	3.24	2.400	2.662	.284	.682	.756	1.637	2.012	1.814	.150	.422	.390	.062	.177	.160
9′	3.12	30.371	3.26	2.720	2.618	.107	.291	.280	.792	.733	.762	.043	.131	.110	.014	.042	.035
10′	4.25	76.766	3.29	3.040	2.560	.043	.131	.110	.398	.282	.334	0	0	0	0	0	0
					Σ =	8.122 =A	8.727 =B	18.440 =C	14.387 =D	46.556 =E	23.285 =F						

1	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
Point or *a*	col. 16× C_2C_4	col. 17× C_6	col. 18× C_3	col. 19− col. 20	col. 21+ col. 22	H=col. 23÷C_H	col. 24× C_6	col. 16× C_2	col. 25+ col. 26− col. 17	V=col. 27÷C_3	col. 24× C_4	col. 16× C_7	col. 29+ col. 30− col. 17	M_A'= col. 31÷ 0.1C_8	(col. 28× l)−(col. 29×h)	l−(10× col. 5)	M_B'= −col. 32 −col. 33 +col. 34
0_1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0_2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0_3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0_4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1′	59.088	−45.088	−103.451	104.176	.725	.063	−.219	8.306	−4.903	.978	.448	12.753	+.251	+.46	+29.73	+30.40	+.21
2′	44.411	−35.432	−77.875	79.843	1.968	.172	−.597	6.243	−4.562	.911	1.224	9.586	+.602	+1.29	+24.80	+27.20	+1.11
3′	30.804	−26.008	−54.068	56.812	2.744	.240	−.833	4.330	−3.996	.798	1.707	6.649	+.863	+1.85	+19.46	+24.00	+2.69
4′	19.062	−17.261	−33.447	36.323	2.876	.251	−.872	2.680	−3.163	.632	1.786	4.114	+.927	+1.99	+13.87	+20.80	+4.94
5′	10.077	−9.920	−17.635	19.997	2.362	.206	−.716	1.417	−2.157	.431	1.465	2.175	+.786	+1.69	+8.58	+17.60	+7.33
6′	4.286	−4.662	−7.450	8.948	1.498	.131	−.455	.603	−1.195	.239	.932	.925	+.514	+1.10	+4.33	+14.40	+8.97
7′	1.536	−1.840	−2.640	3.376	.736	.064	−.222	.216	−.536	.107	.455	.331	+.256	+.55	+1.80	+11.20	+8.85
8′	.474	−.614	−.802	1.089	.287	.025	−.087	.067	−.197	.039	.178	.102	+.103	+.22	+.62	+8.00	+7.16
9′	.107	−.146	−.175	.253	.078	.007	−.024	.015	−.051	.010	.050	.023	+.031	+.07	+.15	+4.80	+4.58
10′	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	+1.60	+1.60

$l=32.00$ ft. $h=25.30$ ft $dx=3.20$ ft.

$$C_1=\frac{C}{B}=+2.113 \qquad C_6=\frac{CB}{A}-F=-3.471$$

$$C_2=\frac{B}{A}=+1.074 \qquad C_7=\frac{D}{B}=+1.649$$

$$C_3=\frac{B^2}{A}-D=-5.010 \qquad C_8=\frac{AD}{B}-B=+4.663$$

$$C_4=\frac{DC}{B}-F=+7.114 \qquad C_2C_4=+7.640$$

$$C_5=\frac{FC}{B}-E=+2.645 \qquad C_H=C_3C_5-C_6C_4=+11.442$$

Table 11 (c).—*Calculation of moments, vertical reactions, and thrusts*

Expression	Value, span 1	Value, span 2
$H_1=\frac{(lC_1-h)100C_3-(10C_2-l)100C_4}{-10{,}000C_H}=$	0.0802	0.0531
$V_1=\frac{H_1(100C_6)+10C_2-l}{100C_3}=$	.0562	.0793
$M_{1A}=\frac{H_1(100C_4)+10C_7-l}{10C_8}=$	.606	.478
$M_{1B}=M_{1A}+V_1l-H_1h=$	1.703	1.669
$H_2=\frac{h(100C_3)}{10{,}000C_H}=$	.216	.111
$V_2=H_2\left(\frac{C_6}{C_3}\right)=$	.0605	.0769
$M_{2A}=H_2\left(\frac{100C_4}{10C_8}\right)=$	3.036	1.693
$M_{2B}=M_{2A}+V_2l-H_2h=$	1.610	1.346

Table 11 (d).—*Calculation of arch rib joint constants*

Expression	Value, span 1	Value, span 2
Moment stiffness $=M_{1B}\div 12=$	0.142	0.139
Induced thrust $=H_1\div M_{1B}=$	.047	.032
Thrust stiffness $=H_2\div 12h=$	.00090	.00037
Induced moment (at pier top) $=M_{2B}\div H_2=$	7.45	12.13
Induced moment (at footing) $=-M_{2A}\div H_2=$	−14.06	−15.25
Moment carry-over $=-M_{1A}\div M_{1B}=$	−.35	−.29

Table 12.—*Pier constants*

$l=22.5$ ft. $t=2.0$ ft.

$I=t^3/12=0.667$

Expression	Value
Moment stiffness $=4I\div l=$	0.119
Induced thrust $=3\div 2l=$	.067
Thrust stiffness $=12I\div l^3=$	.00070
Induced moment $=l\div 2=$	11.25
Moment carry-over $=$	+.50

Table 13.—*Distribution of moment and thrust stiffness*

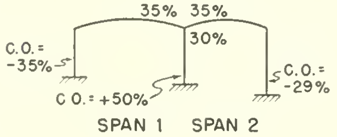
Member	Moment stiffness		Thrust stiffness	
	Value	Distribution factor	Value	Distribution factor
		Percent		*Percent*
Arch rib, span 1	0.142	35	0.00090	45
Arch rib, span 2	.139	35	.00037	19
Pier	119	30	.00070	36
Total	400	100	.00197	100

Table 14.—*Distribution of induced moments, induced thrusts, and moment carry-overs*

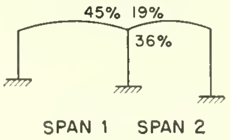
Member	Induced moment—		Induced thrust (multiply value by distributed moments)	Moment carry-over
	At pier top (multiply value by distributed thrusts)	At footing (multiply value by distributed thrusts)		
				Percent
Arch rib, span 1	+7.45	−14.06	+0.046	−35
Arch rib, span 2	+12.13	−15.25	+.032	−29
Pier	−11.25	+11.25	−.067	+50

Tables 15 & 16.—*Unit moment and thrust distributions, unit loads in span 1*

MOMENT DISTRIBUTION FACTORS



THRUST DISTRIBUTION FACTORS



Induced Thrusts

0.046×moment (arch rib, span 1)
0.032×moment (arch rib, span 2)
−0.067×moment (pier)

Induced Moments

7.44 (−14.06 at footing)×thrust (arch rib, span 1)
12.13 (−15.25 at footing)×thrust (arch rib, span 2)
−11.25 (−11.25 at footing)×thrust (pier)

Table 15.—*Distribution of unit moment, span 1*

	Rib	Pier	Rib		Rib	Pier	Rib	
Fixed-end moment	−1.000	0	0		0	0	0	Fixed-end thrust
Balancing moment	+.350	+.300	+.350	Induced thrust →	+.016	−.020	+.011	
	−.022	+.034	−.012	←Induced moment	−.003	−.003	−.001	Balancing thrust
Balancing moment	------	-------	------	Induced thrust→	------	------	------	
	------	-------	------	←Induced moment	------	------	------	Balancing thrust
Final moments	−.672	+.334	+.338		+.013	−.023	+.010	Final thrusts

	Rib	Pier	Rib	Rib	Pier	Rib	
Sum of balancing thrusts	−0.003	−0.003	−0.001	+0.042	+0.034	+0.015	Moment induced at footing
Sum of balancing moments	+.350	+.300	+.350	−.123	+.150	−.102	Moment carried over to footing
	Total moment at footing			−.081	+.184	−.087	

Footing moments

Table 16.—*Distribution of unit thrust, span 1*

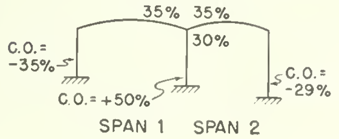
	Rib	Pier	Rib		Rib	Pier	Rib	
Fixed-end moment	0	0	0		+1.000	0	0	Fixed-end thrust
	−3.348	+4.050	−2.305	←Induced moment	−.450	−.360	−.190	Balancing thrust
Balancing moment	+.561	+.481	+.561	Induced thrust→	+.026	−.032	+.018	
	−.037	+.045	−.036	←Induced moment	−.005	−.004	−.003	Balancing thrust
Balancing moment	+.010	+.008	+.010	Induced thrust→	------	------	------	
Final moments	−2.814	+4.584	−1.770		+.571	−.396	−.175	Final thrusts

	Rib	Pier	Rib	Rib	Pier	Rib	
Sum of balancing thrusts	−0.455	−0.364	−0.193	+6.397	+4.095	+2.943	Moment induced at footing
Sum of balancing moments	+.571	+.489	+.571	−.200	+.245	−.166	Moment carried over to footing
	Total moment at footing			+6.197	+4.340	+2.777	

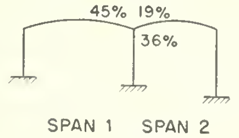
Footing moments

Tables 17 & 18.—*Unit moment and thrust distributions, unit loads in span 2*

MOMENT DISTRIBUTION FACTORS



THRUST DISTRIBUTION FACTORS



INDUCED THRUSTS

+0.046×moment (arch rib, span 1)
+0.032×moment (arch rib, span 2)
−0.067×moment (pier)

INDUCED MOMENTS

+7.44 (−14.06 at footing)×thrust (arch rib, span 1)
+12.13 (−15.25 at footing)×thrust (arch rib, span 2)
−11.25 (−11.25 at footing)×thrust (pier)

Table 17.—*Distribution of unit moment, span 2*

	Rib	Pier	Rib		Rib	Pier	Rib	
Fixed-end moment	0	0	+1.000		0	0	0	Fixed-end thrust
Balancing moment	−.350	−.300	−.350	Induced thrust→	−.016	+.020	−.011	
	+.022	−.034	+.012	←Induced moment	+.003	+.003	+.001	Balancing thrust
Balancing moment	------	------	------	Induced thrust→	------	------	------	
	------	------	------	←Induced moment	------	------	------	Balancing thrust
Final moments	−.328	−.334	+.662		−.013	+.023	−.010	Final thrusts

	Rib	Pier	Rib	Rib	Pier	Rib	
Sum of balancing thrusts	+0.003	+0.003	+0.001	−0.042	−0.034	−0.015	Moment induced at footing
Sum of balancing moments	−.350	−.300	−.350	+.123	−.150	+.102	Moment carried over to footing
	Total moment at footing			+.081	−.184	+.087	

Footing moments

Table 18.—*Distribution of unit thrust, span 2*

	Rib	Pier	Rib		Rib	Pier	Rib	
Fixed-end moment	0	0	0		0	0	−1.000	Fixed-end thrust
	+3.348	−4.050	+2.305	←Induced moment	+.450	+.360	+.190	Balancing thrust
Balancing moment	−.561	−.481	−.561	Induced thrust→	−.026	+.032	−.018	
	+.037	−.045	+.036	←Induced moment	+.005	+.004	+.003	Balancing thrust
Balancing moment	−.010	−.008	−.010	Induced thrust→	------	------	------	
inal moments	+2.814	−4.584	+1.770		+.429	+.396	−.825	Final thrusts

	Rib	Pier	Rib	Rib	Pier	Rib	
Sum of balancing thrusts	+0.455	+0.364	+0.193	−6.397	−4.095	−2.943	Moment induced at footing
Sum of balancing moments	−.571	−.489	−.571	+.200	−.245	+.166	Moment carried over to footing
	Total moment at footing			−6.197	−4.340	−2.777	

Footing moments

Table 19.—*Tabulation of final moments and thrusts, span 1*

Table 19 (a)

Fixed-end moment = −1.000				
M_B	$M_{B'}$	H_1	H_2	H_3
−0.672	+0.338	+0.013	−0.023	+0.010

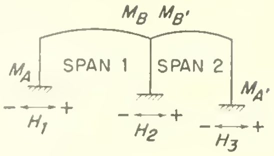


Table 19 (b)

Fixed-end thrust = +1.000				
M_B	$M_{B'}$	H_1	H_2	H_3
−2.814	−1.770	+0.571	−0.396	−0.175

Table 19 (c)

Unit load at point—	Fixed-end moment	Fixed-end thrust	M and H due to fixed-end moment					M and H due to fixed-end thrust					Final values of M and H				
			M_B	$M_{B'}$	H_1	H_2	H_3	M_B	$M_{B'}$	H_1	H_2	H_3	M_B	$M_{B'}$	H_1	H_2	H_3
1	−0.06	+0.137	−0.04	+0.02	+0.001	−0.001	+0.001	−0.39	−0.24	+0.078	−0.054	−0.024	−0.43	−0.22	+0.079	−0.055	−0.023
2	−.63	+.374	−.42	+.21	+.008	−.014	+.006	−1.05	−.66	+.214	−.148	−.065	−1.47	−.45	+.222	−.162	−.059
3	−2.07	+.545	−1.39	+.70	+.027	−.048	+.021	−1.53	−.96	+.311	−.215	−.095	−2.92	−.26	+.338	−.263	−.074
4	−4.69	+.507	−3.15	+1.59	+.061	−.108	+.047	−1.71	−1.07	+.347	−.240	−.106	−4.86	+.52	+.408	−.348	−.059
5	−8.06	+.536	−5.42	+2.72	+.105	−.185	+.081	−1.51	−.95	+.306	−.212	−.094	−6.93	+1.77	+.411	−.397	−.013
6	−10.87	+.376	−7.30	+3.67	+.141	−.250	+.109	−1.06	−.67	+.215	−.149	−.066	−8.36	+3.00	+.356	−.399	+.043
7	−11.66	+.206	−7.84	+3.94	+.152	−.268	+.117	−.58	−.36	+.118	−.082	−.036	−8.42	+3.58	+.270	−.350	+.081
8	−10.06	+.086	−6.76	+3.40	+.131	−.231	+.101	−.24	−.15	+.049	−.034	−.015	−7.00	+3.25	+.180	−.265	+.086
9	−6.70	+.023	−4.50	+2.26	+.087	−.154	+.067	−.06	−.04	+.013	−.009	−.004	−4.56	+2.22	+.100	−.163	+.063
10	−2.41	0	−1.62	+.81	+.031	−.055	+.024	0	0	0	0	0	−1.62	+.81	+.031	−.055	+.024

Table 20.—*Tabulation of final moments and thrusts, span 2*

Table 20 (a)

Fixed-end moment = +1.000				
M_B	$M_{B'}$	H_1	H_2	H_3
−0.328	+0.662	−0.013	+0.023	−0.010

Table 20 (b)

Fixed-end thrust = −1.000				
M_B	$M_{B'}$	H_1	H_2	H_3
+2.814	+1.770	+0.429	+0.396	−0.825

Table 20 (c)

Unit load at point—	Fixed-end moment	Fixed-end thrust [1]	M and H due to fixed-end moment					M and H due to fixed-end thrust					Final values of M and H				
			M_B	$M_{B'}$	H_1	H_2	H_3	M_B	$M_{B'}$	H_1	H_2	H_3	M_B	$M_{B'}$	H_1	H_2	H_3
10′	+1.60	0	−0.52	+1.06	−0.021	+0.037	−0.016	0	0	0	0	0	−0.52	+1.06	−0.021	+0.037	−0.016
9′	+4.58	−.007	−1.52	+3.03	−.060	+.105	−.046	+.02	+.01	+.003	+.003	−.006	−1.48	+3.04	−.057	+.108	−.052
8′	+7.16	−.025	−2.35	+4.74	−.093	+.165	−.072	+.07	+.04	+.011	+.010	−.021	−2.28	+4.78	−.082	+.175	−.093
7′	+8.85	−.064	−2.90	+5.86	−.115	+.204	−.089	+.18	+.11	+.027	+.025	−.053	−2.72	+5.97	−.088	+.229	−.142
6′	+8.97	−.131	−2.94	+5.94	−.117	+.206	−.090	+.37	+.23	+.056	+.052	−.108	−2.57	+6.17	−.061	+.758	−.198
5′	+7.33	−.206	−2.40	+4.85	−.095	+.169	−.073	+.58	+.36	+.088	+.082	−.170	−1.82	+5.21	−.007	+.251	−.243
4′	+4.94	−.251	−1.62	+3.27	−.164	+.114	−.049	+.71	+.44	+.108	+.099	−.207	−.91	+3.71	+.044	+.213	−.256
3′	+2.69	−.240	−.88	+1.78	−.035	+.062	−.027	+.68	+.42	+.103	+.095	−.198	−.20	+2.20	+.068	+.157	−.225
2′	+1.11	−.172	−.36	+.73	−.014	+.026	−.011	+.48	+.30	+.074	+.068	−.142	+.12	+1.03	+.060	+.094	−.153
1′	+.21	−.163	−.07	+.14	−.003	+.005	−.002	+.18	+.11	+.027	+.025	−.052	+.11	+.25	+.024	+.030	−.054

[1] Negative sign used to conform to sign convention.

Table 21.—*Tabulation of final moments at footings, span 1*

Table 21 (a)

Fixed-end moment = −1.000		
M_1	M_2	M_3
−0.081	+0.184	−0.087

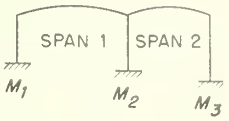


Table 21 (b)

Fixed-end thrust = +1.000		
M_1	M_2	M_3
+6.197	+4.340	+2.777

Table 21 (c)

Unit load at point	Fixed-end moment (M_B)	Fixed-end thrust	M_A	M due to fixed-end moment			M due to fixed-end thrust			Final values of M		
				M_1	M_2	M_3	M_1	M_2	M_3	M_1+M_A	M_2	M_3
1	−0.06	+0.137	−0.77	−0.005	+0.01	−0.005	+0.85	+0.59	+0.38	+0.07	+0.60	+0.37
2	−.63	+.374	−2.16	−.05	+.12	−.06	+2.32	+1.62	+1.04	+.11	+1.74	+.98
3	−2.07	+.545	−3.24	−.17	+.38	−.18	+3.38	+2.37	+1.51	−.03	+2.75	+1.33
4	−4.69	+.607	−3.75	−.38	+.86	−.41	+3.76	+2.63	+1.69	−.37	+3.49	+1.28
5	−8.06	+.536	−3.44	−.65	+1.48	−.70	+3.32	+2.33	+1.49	−.77	+3.81	+.79
6	−10.87	+.376	−2.52	−.88	+2.00	−.95	+2.33	+1.63	+1.04	−1.07	+3.63	+.09
7	−11.66	+.206	−1.45	−.94	+2.15	−1.01	+1.28	+.89	+.57	−1.11	+3.04	−.44
8	−10.06	+.086	−.63	−.81	+1.85	−.88	+.53	+.37	+.24	−.91	+2.22	−.64
9	−6.70	+.023	−.17	−.54	+1.23	−.58	+.14	+.10	+.06	−.57	+1.33	−.52
10	−2.41	0	0	−.20	+.44	−.21	0	0	0	−.20	+.44	−.21

Table 22.—*Tabulation of final moments at footings, span 2*

Table 22 (a)

Fixed-end moment = +1.000		
M_1	M_2	M_3
+0.081	−0.184	+0.087

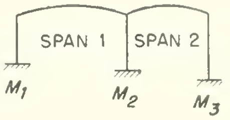


Table 22 (b)

Fixed-end thrust = −1.000		
M_1	M_2	M_3
−6.197	−4.340	−2.777

Table 22 (c)

Unit load at point	Fixed-end moment (M_B')	Fixed-end thrust	M_A'	M due to fixed-end moment			M due to fixed-end thrust			Final values of M		
				M_1	M_2	M_3	M_1	M_2	M_3	M_1	M_2	M_3+M_A'
10′	+1.60	0	0	+0.13	−0.29	+0.14	0	0	0	+0.13	−0.29	+0.14
9′	+4 58	−.007	+.07	+.37	−.84	+.40	−.04	−.03	−.02	+.33	−.87	+.45
8′	+7.16	−.025	+.22	+.58	−1.32	+.62	−.15	−.11	−.07	+.43	−1.43	+.77
7′	+8.85	−.064	+.55	+.72	−1.63	+.77	−.40	−.28	−.18	+.32	−1.91	+1.14
6′	+8.97	−.131	+1.10	+.73	−1.65	+.78	−.81	−.57	−.36	−.08	−2.22	+1.52
5′	+7.33	−.206	+1.69	+.59	−1.35	+.64	−1.28	−.89	−.57	−.69	−2.24	+1.76
4′	+4.94	−.251	+1.99	+.40	−.91	+.43	−1.56	−1.09	−.70	−1.16	−2.00	+1.72
3′	+2.69	−.240	+1.85	+.22	−.49	+.23	−1.49	−1.04	−.67	−1.27	−1.53	+1.41
2′	+1.11	−.172	+1.29	+.09	−.20	+.10	−1.07	−.75	−.48	−.98	−.95	+.91
1′	+.21	−.063	+.46	+.02	−.04	+.02	−.40	−.27	−.17	−.38	−.31	+.31

New Publications

THE IDENTIFICATION OF ROCK TYPES

To meet popular demand a convenient 6 x 9-inch reprint has been made of the article *The Identification of Rock Types,* by D. O. Woolf, which appeared in Public Roads, vol. 26, No. 2, June 1950. The article presents a simple method for use by the highway engineer in making field identification of the different types of rock with which he is concerned. It will be extremely useful to engineers, engineering students, and others whose work requires a limited, practical knowledge of geology. The reprint is for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 10 cents a copy.

A BIBLIOGRAPHY OF HIGHWAY PLANNING REPORTS

The Bureau of Public Roads recently published a 48-page *Bibliography of Highway Planning Reports,* which is now for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 30 cents a copy. The bibliography covers the period 1930 to date, and includes listings of Nation-wide, State, and city highway planning reports such as those of State-wide highway planning surveys and of traffic, origin-destination, design, and highway needs studies. The reports range from long-term studies of State-wide scope to discussions and plans for individual routes, and are the work of the Bureau of Public Roads and State, city, and consulting engineers.

The interest in highway planning continues to increase. This bibliography makes available a listing of reports on the subject, useful both to those interested in the general field of planning and to those concerned with a particular State, city, or route.

COMPARATIVE EFFECT OF HINGED AND FIXED FOOTINGS AT CRITICAL POINTS

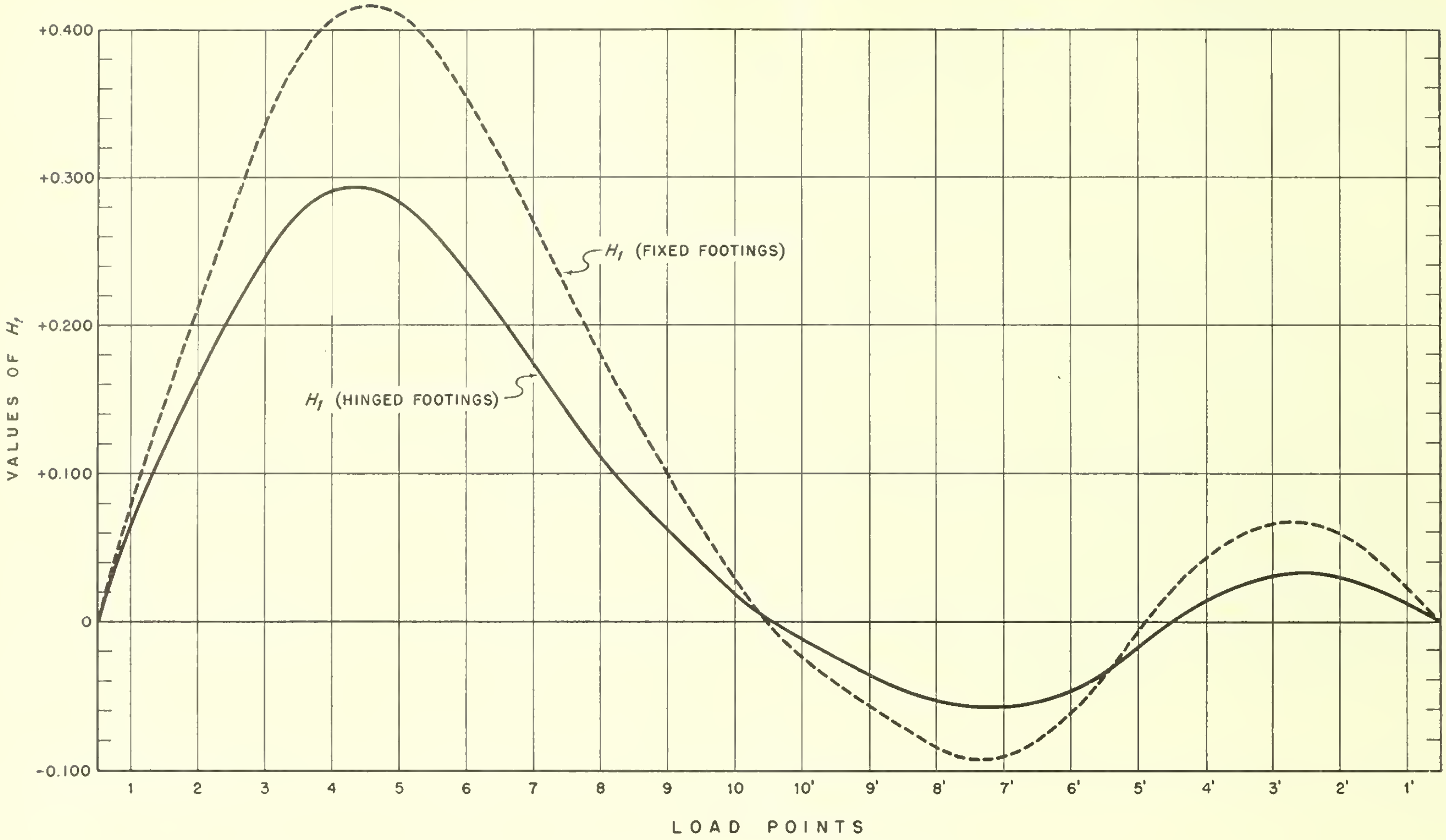


Figure 13.—Influence line for H_1.

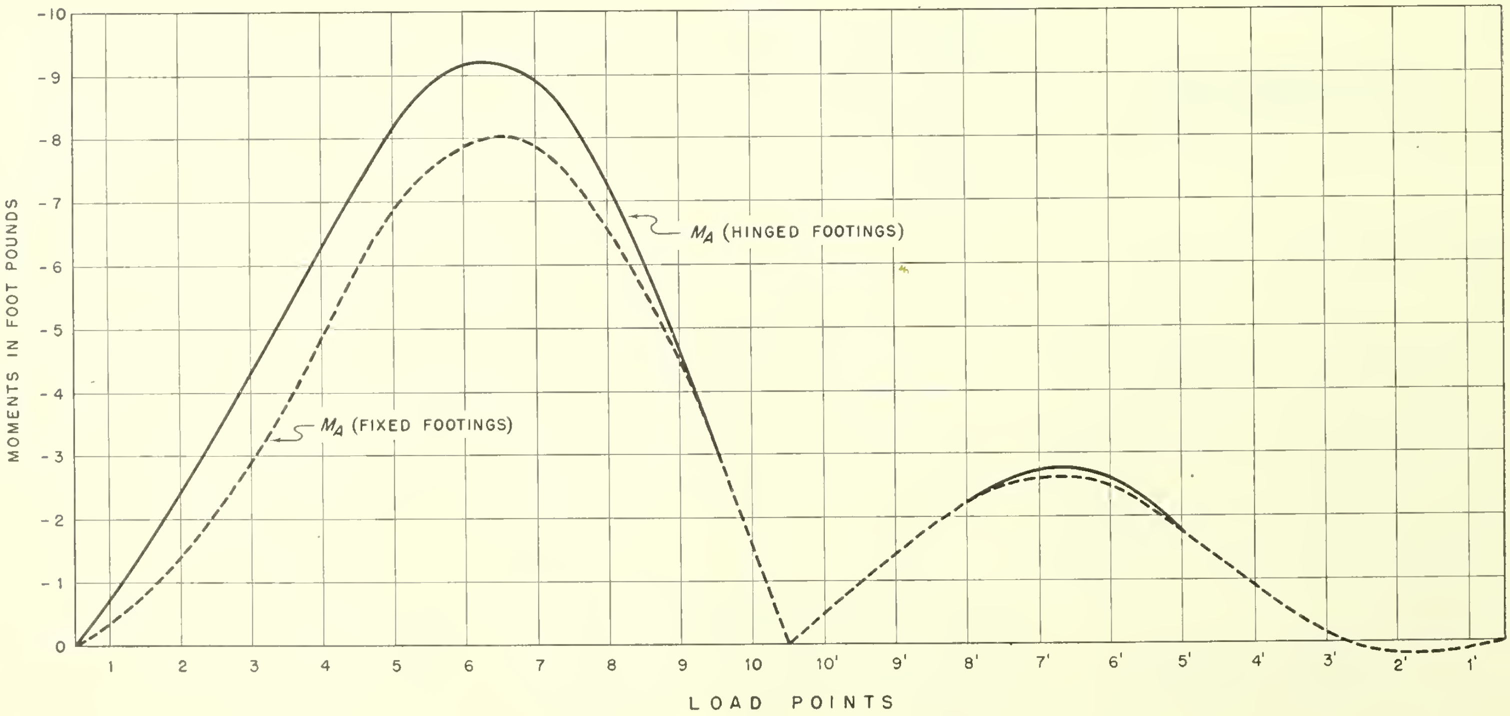


Figure 14.—Influence line for M_A.

U. S. GOVERNMENT PRINTING OFFICE: 1950

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in Public Roads, may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

PUBLICATIONS
# of the Bureau of Public Roads

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(See also adjacent column)

Reports of the Chief of the Bureau of Public Roads:
1937, 10 cents. 1938, 10 cents. 1939, 10 cents.

Work of the Public Roads Administration:
1940, 10 cents. 1942, 10 cents. 1948, 20 cents.
1941, 15 cents. 1946, 20 cents. 1949, 25 cents.
1947, 20 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.
Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.
Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.
Part 4 . . . Official Inspection of Vehicles. 10 cents.
Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.
Part 6 . . . The Accident-Prone Driver. 10 cents.

UNIFORM VEHICLE CODE

Act I.—Uniform Motor-Vehicle Administration, Registration, Certificate of Title, and Antitheft Act. 10 cents.
Act II.—Uniform Motor-Vehicle Operators' and Chauffeurs' License Act. 10 cents.
Act III.—Uniform Motor-Vehicle Civil Liability Act. 10 cents.
Act IV.—Uniform Motor-Vehicle Safety Responsibility Act. 10 cents.
Act V.—Uniform Act Regulating Traffic on Highways. 20 cents.
Model Traffic Ordinance. 15 cents.

MISCELLANEOUS PUBLICATIONS

Bibliography of Highway Planning Reports. 30 cents.
Construction of Private Driveways (No. 272MP). 10 cents.
Economic and Statistical Analysis of Highway Construction Expenditures. 15 cents.
Electrical Equipment on Movable Bridges (No. 265T). 40 cents.
Federal Legislation and Regulations Relating to Highway Construction. 40 cents.
Financing of Highways by Counties and Local Rural Governments, 1931–41. 45 cents.
Guides to Traffic Safety. 10 cents.
Highway Accidents. 10 cents.
Highway Bridge Location (No. 1486D). 15 cents.
Highway Capacity Manual. 65 cents.
Highway Needs of the National Defense (House Document No. 249). 50 cents.
Highway Practice in the United States of America. 50 cents.
Highway Statistics, 1945. 35 cents.
Highway Statistics, 1946. 50 cents.
Highway Statistics, 1947. 45 cents.
Highway Statistics, 1948. 65 cents.
Highway Statistics, Summary to 1945. 40 cents.
Highways of History. 25 cents.
Identification of Rock Types. 10 cents.
Interregional Highways (House Document No. 379). 75 cents.
Legal Aspects of Controlling Highway Access. 15 cents.
Manual on Uniform Traffic Control Devices for Streets and Highways. 50 cents.
Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft. $1.50.
Public Control of Highway Access and Roadside Development. 35 cents.
Public Land Acquisition for Highway Purposes. 10 cents.
Roadside Improvement (No. 191MP). 10 cents.
Specifications for Construction of Roads and Bridges in National Forests and National Parks (FP–41). $1.25.
Taxation of Motor Vehicles in 1932. 35 cents.
The Local Rural Road Problem. 20 cents.
Tire Wear and Tire Failures on Various Road Surfaces. 10 cents.
Transition Curves for Highways. $1.25.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

ANNUAL REPORTS

(See also adjacent column)

Public Roads Administration Annual Reports:
1943. 1944. 1945.

MISCELLANEOUS PUBLICATIONS

Bibliography on Automobile Parking in the United States.
Bibliography on Highway Lighting.
Bibliography on Highway Safety.
Bibliography on Land Acquisition for Public Roads.
Bibliography on Roadside Control.
Express Highways in the United States: a Bibliography.
Indexes to Public Roads, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF AUGUST 31, 1950

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$13,306	$12,611	$6,293	389.7	$4,145	$1,971	101.8	$13,069	$6,729	319.6	$29,825	$14,993	811.1
Arizona	724	3,447	2,410	75.8	1,094	732	20.8	6,246	4,478	118.0	10,787	7,620	214.6
Arkansas	1,775	8,901	5,109	213.2	6,390	3,101	190.7	17,192	8,472	435.1	32,483	16,682	839.0
California	3,115	30,914	12,388	194.2	4,735	2,395	57.0	39,106	19,244	247.5	74,755	34,027	498.7
Colorado	2,728	3,931	2,115	61.2	2,580	1,459	83.3	15,227	8,777	295.4	21,738	12,351	439.9
Connecticut	2,162	9,059	4,312	24.6	2,011	1,217	3.2	7,236	4,106	12.4	18,306	9,635	40.2
Delaware	1,450	1,676	847	22.4	1,391	694	18.5	5,567	2,669	54.6	8,634	4,210	95.5
Florida	4,068	15,795	7,962	441.3	6,150	3,248	162.9	11,566	5,825	273.4	33,511	17,035	877.6
Georgia	1,390	18,799	9,642	500.2	7,875	3,934	191.4	35,937	16,926	740.7	62,611	30,502	1,432.3
Idaho	4,221	8,499	5,353	293.1	2,698	1,008	90.5	6,513	4,134	189.6	17,710	10,495	573.2
Illinois	20,302	42,514	22,436	369.4	11,218	5,614	110.6	53,161	25,548	370.1	106,893	53,598	850.1
Indiana	13,818	20,456	10,498	101.3	6,576	3,456	44.8	14,993	7,784	82.3	42,025	21,738	228.4
Iowa	3,280	11,016	4,072	392.9	4,910	1,771	232.4	20,916	10,258	790.2	36,842	16,101	1,415.5
Kansas	4,376	6,492	3,115	907.6	7,492	3,771	561.3	12,907	6,597	546.7	26,891	13,483	2,015.6
Kentucky	1,796	13,191	6,427	143.6	6,510	3,214	140.5	18,588	9,159	336.1	38,289	18,800	620.2
Louisiana	4,318	20,257	8,942	177.8	8,634	4,213	109.6	17,810	9,267	203.6	46,701	22,422	491.0
Maine	1,692	7,964	4,194	105.4	807	514	5.5	6,965	3,557	81.7	15,736	8,265	192.6
Maryland	1,620	7,897	3,698	29.2	2,206	921	19.5	17,028	8,243	61.9	27,131	12,862	110.6
Massachusetts	2,651	7,041	2,721	6.6	9,703	4,925	9.8	61,257	29,918	62.7	78,001	37,564	79.1
Michigan	4,788	15,289	7,817	437.0	9,613	4,998	301.5	43,320	17,991	369.5	68,222	30,806	1,108.0
Minnesota	1,779	9,877	5,623	880.5	4,416	2,216	383.4	23,834	12,717	883.3	38,127	20,556	2,147.2
Mississippi	5,416	15,499	7,817	621.1	1,483	743	61.8	7,351	3,755	241.9	24,333	12,315	924.8
Missouri	6,767	29,156	15,968	754.2	5,965	2,576	129.4	32,343	16,053	620.7	67,464	34,597	1,504.3
Montana	5,199	14,876	7,700	493.2	2,800	1,661	77.4	15,064	9,386	482.5	32,740	18,747	1,053.1
Nebraska	4,312	17,693	9,249	601.5	5,521	2,512	105.3	12,852	7,353	305.0	36,066	19,114	1,011.8
Nevada	1,328	4,238	3,493	139.0	1,262	1,043	42.1	5,048	4,123	192.3	10,548	8,659	373.4
New Hampshire	1,715	5,502	2,686	46.3	608	298	7.3	3,010	1,455	23.8	9,120	4,439	77.4
New Jersey	3,020	4,195	1,985	7.9	1,058	529	3.8	20,218	9,643	27.7	25,471	12,157	39.4
New Mexico	1,539	7,242	4,620	247.3	3,053	1,967	79.7	6,320	4,131	214.1	16,615	10,718	541.1
New York	27,498	78,306	38,466	225.8	21,959	8,936	32.3	93,182	45,114	187.5	193,447	92,516	445.6
North Carolina	1,516	21,608	10,395	537.9	5,105	2,437	170.0	22,094	10,526	558.3	48,807	23,358	1,266.2
North Dakota	3,080	8,473	4,385	1,242.7	2,816	1,413	250.8	8,903	4,436	599.1	20,192	10,234	2,092.6
Ohio	9,485	41,567	20,114	376.9	14,753	6,753	133.6	54,043	26,873	256.0	110,363	53,740	766.5
Oklahoma	1,176	14,688	8,332	173.0	13,682	6,650	303.0	17,332	8,355	423.7	45,702	23,337	899.7
Oregon	787	5,258	3,022	64.2	2,710	1,530	37.0	11,939	6,783	184.5	19,907	11,335	285.7
Pennsylvania	5,364	23,620	11,721	63.4	19,413	9,680	43.9	82,147	40,620	196.1	125,180	62,021	303.4
Rhode Island	184	6,848	3,422	54.7	4,023	2,073	6.3	9,826	4,896	5.3	20,697	10,391	66.3
South Carolina	3,224	8,202	3,995	209.3	1,830	1,008	95.3	9,749	4,948	374.9	19,781	9,951	679.5
South Dakota	1,229	8,848	5,051	859.5	3,944	2,456	316.5	8,768	5,321	751.7	21,560	12,828	1,927.7
Tennessee	1,087	14,818	7,015	308.0	8,195	3,948	196.2	18,105	8,239	362.5	41,118	19,202	866.7
Texas	5,398	4,756	2,493	296.6	13,689	7,313	328.9	52,761	24,471	1,516.2	71,206	34,277	2,141.7
Utah	1,534	4,411	3,233	125.3	1,111	799	56.9	6,591	4,824	173.5	12,113	8,856	355.7
Vermont	998	2,577	1,412	49.1	962	479	13.6	4,640	2,266	36.9	8,179	4,157	99.6
Virginia	4,170	24,908	12,489	579.5	5,146	2,553	173.5	15,399	7,560	259.5	45,453	22,602	1,012.5
Washington	530	14,810	6,601	92.1	2,305	1,055	75.6	20,448	9,386	157.5	37,563	17,042	325.2
West Virginia	1,857	16,248	6,777	161.5	1,649	920	27.0	9,812	4,914	82.5	27,709	12,611	271.0
Wisconsin	8,825	18,217	9,529	298.2	4,383	2,176	208.0	17,629	8,476	426.4	40,229	20,181	932.6
Wyoming	845	2,046	1,353	34.2	1,602	837	34.7	7,310	4,801	263.0	10,958	6,991	331.9
Hawaii	1,114	8,511	3,809	21.8	2,258	998	12.5	2,941	1,454	14.2	13,710	6,261	48.5
District of Columbia	1,399	5,178	2,951	6.0	340	170	.9	647	324	.9	6,165	3,445	7.8
Puerto Rico	856	14,505	6,563	71.2	728	324	2.7	10,024	4,220	37.7	25,257	11,107	111.6
TOTAL	206,811	722,430	362,620	14,528.4	265,507	131,209	5,865.0	1,034,934	517,105	15,450.3	2,022,871	1,010,934	35,843.7

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BUREAU OF PUBLIC ROADS

U. S. DEPARTMENT OF COMMERCE

E. A. STROMBERG, Editor

Contents of this publication may be reprinted. Mention of source is requested.

Traffic Trends on Rural Roads in 1949

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by THOMAS B. DIMMICK, Head,
Current Data Analysis Unit

Total travel on rural roads in 1949 broke all records, exceeding the 1948 previous high by 7 percent and the prewar peak by 27 percent. On the 350,000 miles of main rural roads in the United States, travel in 1949 was over 159 billion vehicle-miles, of which 78 percent was by passenger cars, 1 percent by busses, and 21 by freight-carrying vehicles.

Trucks and combinations hauled 7 percent more ton-mileage of freight in 1949 than in 1948 and 52 percent more than in 1941, the increase resulting largely from greater use of heavier vehicles. Truck-combination travel was 10 percent higher than in 1948, 85 percent higher than in 1941, and 244 percent higher than in 1936. Comparable figures for single-unit trucks were 5, 27, and 92 percent. The average carried load for all trucks and combinations in 1949 was 2, 40, and 76 percent above the averages in 1948, 1941, and 1936, respectively.

In 1949 more than 5 percent of all trucks and combinations exceeded a State legal weight limit, and 16 percent of the combinations were illegally overloaded in some particular. In comparison with 1948, the percentage of overweight vehicles decreased slightly in 1949 except in the western States.

MOTOR-VEHICLE TRAVEL in 1949 broke all previous records for the fourth consecutive year. The 1949 traffic on all rural roads was 7 percent higher than in 1948, 16 percent higher than in 1947, and about 27 percent above the 1946 volume and the 1941 prewar peak. Geographically the increases over 1948 ranged from 4 percent in the western States to 9 percent in the eastern States, with an average increase in the central States of 7 percent. The largest increase in any of the United States census regions[1] was 12 percent in the West North Central region and the smallest increase was 1 percent in the Pacific region. Records from about 800 automatic traffic recorders, operated continuously throughout the year at permanent stations on main and local roads in all States, were used generally to establish these trends. More extensive traffic surveys, made by a number of States, yielded valuable information concerning the total volume of rural traffic within their boundaries. Consideration has been given to all such available data in this analysis. Where States have prepared and submitted vehicle-mile travel estimates of their own, these have been employed rather than estimates made by applying trend factors.

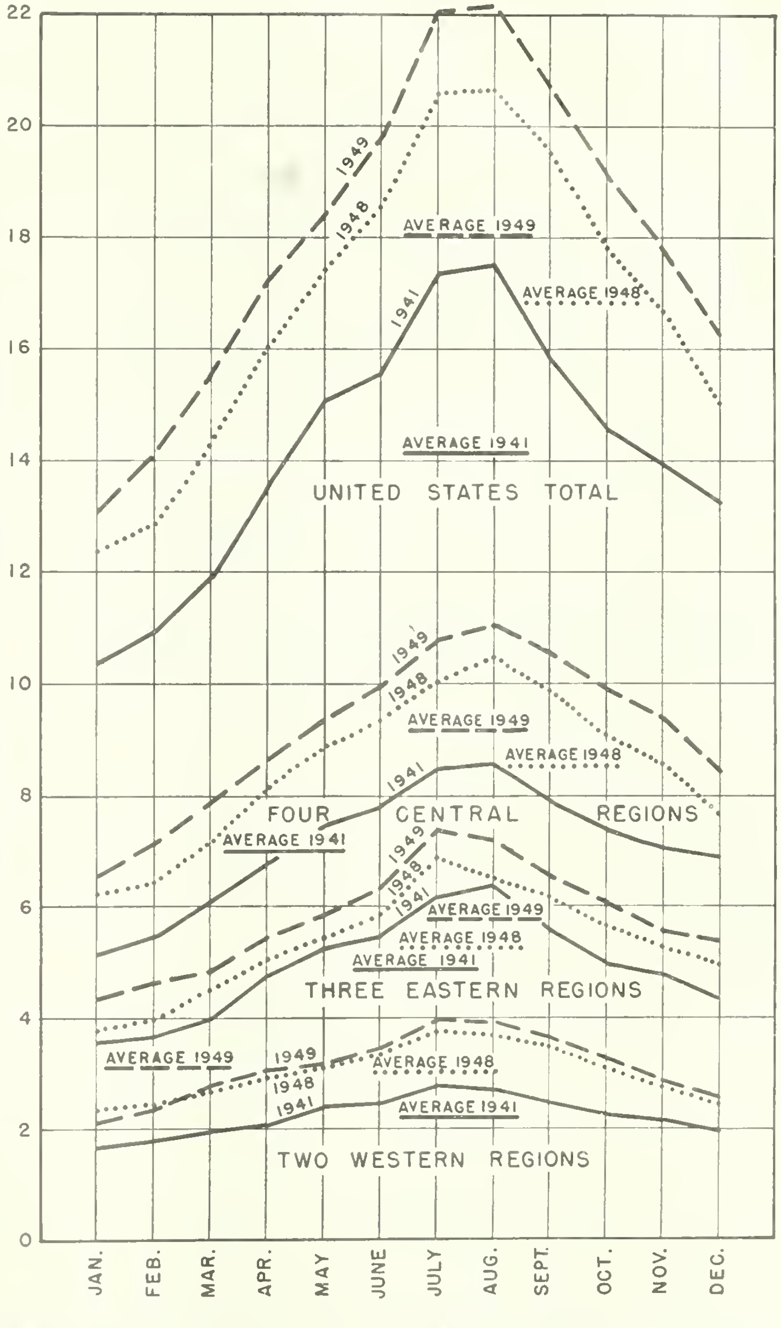

Figure 1.—Travel on all rural roads in 1941, 1948, and 1949, by months.

The variation in travel on rural roads in three main geographic divisions and in the United States as a whole is illustrated in figure 1 for the years 1949, 1948, and 1941, the latter being the prewar peak year. Travel in each month of 1949 in the eastern and central regions and in the United States as a whole was well above that of the corresponding month of the earlier years. The western regions, however, showed only a slight increase from 1948 to 1949.

Summer travel in 1949 reached its prewar importance for the first time since the end of hostilities in 1945. In the last two prewar years the average monthly travel in July and August was 23 percent above that of the average month of the year. Restrictions placed on nonessential driving in 1942 reduced travel in the summer months so that only 13 or 14 percent more traffic used the roads in the summer months than in the average month of the war years. Following the war this seasonal travel increased each succeeding year. Not until 1949, however, did vacation and other summer driving again reach the prewar level, 23 percent above the annual average.

---

[1] The States comprising each census region are indicated in table 1.

1949 Summer Loadometer Survey

The large number of automatic traffic recorders operated on the rural roads of each State gives a good indication of the trend of total traffic on those highways but provides no indication concerning the classification of vehicles by type, weight, or other characteristics. During certain prewar years, generally 1936 and 1937, nearly every State conducted a comprehensive survey of traffic in which all vehicles counted were classified. At the same time a large number of trucks and truck combinations were stopped for the gathering of information concerning their weight, dimensions, and other important features.

In order to determine the wartime trend in weights, dimensions, and other characteristics of commercial vehicles, a brief check survey was made in the summer of 1942 at certain typical stations in most States. From the information collected in the two surveys, which were kept strictly comparable, trends were calculated which were used to determine the changes in traffic and vehicle characteristics that had taken place since the comprehensive survey was made. Since 1942, check surveys have been made annually; most of the States have participated in these each year and all have participated at some time.[2] Forty-six States conducted such surveys in 1949.

Classification counts made in numerous States, in addition to those of the summer survey, added valuable information concerning vehicle-type frequencies. In a few States expanded loadometer surveys have furnished more reliable data concerning vehicle types and weights than can be obtained from the trend data alone, and these have been used in the analysis when available.

The stations used in these summer surveys were selected initially to give a representative cross section of traffic on main rural roads. They were operated for one or more 8-hour periods on a weekday, generally from either 6 a. m. to 2 p. m., or from 2 p. m. to 10 p. m. All traffic passing through the stations during the period was counted and classified into the following categories: local passenger cars; foreign (out-of-State) passenger cars; panel and pickup trucks;[3] other two-axle, four-tire trucks; two-axle, six-tire trucks; three-axle trucks; truck-tractor and semitrailer combinations; truck and trailer combinations or truck-tractor semitrailer and trailer combinations; and busses. The combination-type vehicles were further subdivided according to the number of axles of each.

Table 1.—*Survey period, number of stations operated, number of vehicles counted, and number weighed in each State in the special weight surveys, summer of 1949*

Region and State	Survey period	Number of stations	Total vehicles counted	Trucks and truck combinations	
				Counted	Weighed
New England:					
Connecticut	Aug. 1–12	10	32, 760	5, 817	2, 430
Maine	July 21–Aug. 5	10	24, 308	4, 252	2, 018
Massachusetts	Aug. 8–26	10	36, 083	5, 685	1, 925
New Hampshire	Aug. 8–12	5	15, 282	1, 814	496
Rhode Island	July 25–Aug. 1	5	14, 322	2, 400	1, 145
Vermont	Aug. 1–5	5	10, 002	651	651
Subtotal		45	132, 757	20, 619	8, 665
Middle Atlantic:					
New Jersey	Aug. 16–Sept. 8	10	79, 674	14, 873	1, 977
New York	Sept. 19–24	20	39, 380	10, 265	3, 464
Pennsylvania	July 26–Sept. 1	7	24, 066	4, 818	907
Subtotal		37	143, 120	29, 956	6, 348
South Atlantic:					
Delaware	Aug. 11–25	9	26, 819	7, 922	1, 190
Florida	(1)				
Georgia	Aug. 1–30	12	17, 383	4, 384	3, 020
Maryland	Aug. 8–15	10	39, 267	8, 001	1, 156
North Carolina	Aug. 9–30	12	36, 228	6, 311	4, 324
South Carolina	Sept. 12–26	10	17, 162	4, 659	2, 114
Virginia	Aug. 3–19	10	22, 279	5, 099	3, 815
West Virginia	Aug. 9–Sept. 2	9	13, 170	3, 134	1, 307
Subtotal		72	172, 308	39, 510	16, 926
Eastern regions, subtotal		154	448, 185	90, 085	31, 939
East North Central:					
Illinois	Aug. 23–Sept. 7	47	85, 148	17, 143	6, 449
Indiana	Aug. 3–31	20	44, 232	10, 375	4, 155
Michigan	July 22–Aug. 4	8	21, 928	2, 930	1, 028
Ohio	July 26–Aug. 11	10	26, 244	4, 891	1, 045
Wisconsin	Aug. 4–25	10	24, 945	3, 755	1, 285
Subtotal		95	202, 497	39, 094	13, 962
East South Central:					
Alabama	July 13–29	10	11, 594	2, 910	1, 750
Kentucky	June 20–July 6	10	12, 959	2, 925	1, 133
Mississippi	Aug. 15–Sept. 2	15	24, 253	6, 633	2, 615
Tennessee	Aug. 2–16	10	11, 336	3, 011	1, 710
Subtotal		45	60, 142	15, 479	7, 208
West North Central:					
Iowa	July 25–Aug. 5	10	13, 336	2, 427	2, 424
Kansas	Aug. 11–24	10	10, 566	2, 141	1, 095
Minnesota	Apr. 11–Aug. 3	10	29, 075	6, 492	2, 147
Missouri	Aug. 12–Sept. 1	15	70, 079	14, 323	6, 940
Nebraska	July 21–Aug. 17	20	22, 014	4, 744	4, 593
North Dakota	July 21–Sept. 1	15	22, 960	5, 376	2, 043
South Dakota	June 24–Sept. 21	11	8, 956	1, 391	1, 182
Subtotal		91	176, 986	36, 894	20, 424
West South Central:					
Arkansas	July 25–Aug. 5	10	16, 804	5, 519	1, 441
Louisiana	July 18–27	10	10, 015	3, 019	1, 184
Oklahoma	Aug. 15–29	10	14, 414	3, 139	3, 017
Texas	June 15–Sept. 13	17	29, 982	6, 838	1, 632
Subtotal		47	71, 215	18, 515	7, 274
Central regions, subtotal		278	510, 840	109, 982	48, 868
Mountain:					
Arizona	June 6–17	10	9, 010	1, 962	785
Colorado	Aug. 11–29	10	25, 647	4, 161	439
Idaho	July 25–Aug. 10	13	15, 429	2, 886	2, 707
Montana	Aug. 3–Sept. 1	19	16, 344	3, 197	2, 540
Nevada	July 25–Aug. 9	10	6, 693	1, 036	881
New Mexico	July 21–Aug. 4	10	12, 621	2, 739	1, 283
Utah	July 18–Aug. 5	10	16, 112	3, 215	1, 473
Wyoming	(1)				
Subtotal		82	101, 856	19, 196	10, 108
Pacific:					
California	May 31–June 30	20	[2] 71, 484	13, 153	4, 836
Oregon	Aug. 9–19	8	15, 660	2, 893	1, 984
Washington	June 7–Oct. 7	20	99, 978	18, 007	12, 265
Subtotal		48	187, 122	34, 053	19, 085
Western regions, subtotal		130	288, 978	53, 249	29, 193
United States total		562	1, 248, 003	253, 316	110, 000

[1] No survey made.
[2] Passenger cars not counted; figure given is an estimate based on data from other reports.

Most of the weight stations were operated during July, August, and September. Arizona completed its work in June; California started its survey on the last day in May and completed it in June; Texas operated its stations from June to September; Washington conducted its operations from June to October.

The survey period, number of stations operated, number of vehicles counted, and number weighed are shown for each State in table 1. Almost 1¼ million vehicles were counted at all stations during the period of the survey. About one-fifth of these were freight-carrying vehicles, of which almost one-half were weighed.

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[2] See *Traffic trends on rural roads,* by T. B. Dimmick, PUBLIC ROADS, vol. 25, No. 12, Feb. 1950; vol. 25, No. 7, Mar. 1949; vol. 25, No. 3, Mar. 1948; vol. 24, No. 10, Oct.-Nov.-Dec. 1946; and *Amount and characteristics of trucking on rural roads,* by J. T. Lynch and T. B. Dimmick, PUBLIC ROADS, vol. 23, No. 9, July-Aug.-Sept. 1943.

[3] Single-unit trucks with a carrying capacity of less than 1½ tons.

Wherever traffic volume permitted, every truck and truck combination was stopped and weighed. Where this procedure was impracticable all of the less common types were weighed and the common vehicle types were weighed in sufficient numbers to establish their characteristics from the sample. The type of vehicle, whether loaded or empty, the number of axles, and the weight of each axle were recorded. The axle-spacing and total wheelbase length of the heavier vehicles[4] were measured, and the commodity carried and the type of operation—private or for-hire—were recorded. Passenger cars and busses were counted but not stopped for weighing.

Prewar Travel Trend Maintained

Figure 2 shows in chart form the vehicle-mileage of travel on all rural roads, by type of vehicle, for each year from 1936 to 1949, inclusive. It is apparent that the effect of the drastic restriction of travel during the war period, 1942–45, has now been entirely overcome. A straight line from the top of the bar for 1936 to the top of the bar for 1949 passes through the tops of the bars for 1937, 1939, and 1940, and only slightly above the top of the bar for 1938 and slightly below that for 1941. The recession in business activity in 1938 probably accounts for the lessened volume of traffic in that year; and 1941 was a year of exceptional activity in preparation for the war that followed. Altogether, total traffic has recovered completely from the effect of the war and stands, as near as can be determined from the long-time trend, at a level which would have been reached had the war not occurred.

In the case of travel of trucks and truck combinations,[5] the 1949 value fits the 1936–40 trend, projected, almost exactly. For truck combinations alone, the 1936–48 line lies slightly above the tops of the bars for all intervening years. This and other trend data indicate an accelerating growth in traffic by vehicles of this type.

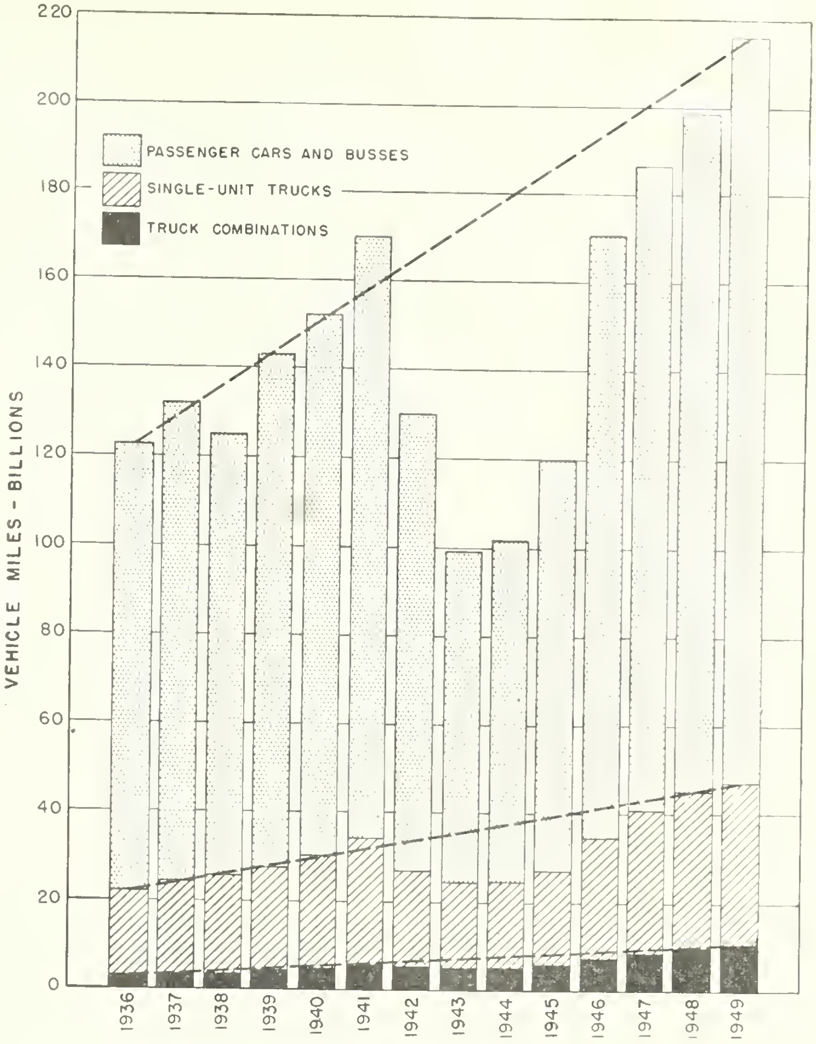

Figure 2.—Travel on all rural roads, 1936-49, by classes of vehicles.

Travel Increases

The ratio of traffic volumes on main rural roads in 1949 to the corresponding volumes in 1948 is shown in table 2. Highways classified under the term "main" include about 350,000 miles and, in general, are those of the entire State systems. In such States as North Carolina, Pennsylvania, and Virginia, where all or a large part of the rural-road mileage is under State control, only the mileage in the primary system is included. The consistent increase in travel on these main highways by most types of vehicles and in all sections of the country is evident in the table. Travel by both local and foreign (out-of-State) passenger cars increased in all regions, but travel by single-unit trucks decreased in the Middle Atlantic, South Atlantic, and Pacific regions and increased only slightly in the New England and West South Central regions. The increases in travel by truck combinations and the decreases by single-unit trucks appear to be a result of the continued shifting to the heavier types.

Use of Truck Combinations

The percentage of travel by vehicle types on main rural roads in 1949 is given in table 3.

Table 2.—*Ratio of 1949 traffic on main rural roads to corresponding traffic in 1948* [1]

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	1.09	1.15	1.08	1.11	1.07	1.01	1.09	1.10	1.08	1.08	1.04	1.05	1.08
Foreign	1.15	1.09	1.15	1.13	1.03	1.33	1.08	1.17	1.10	1.05	1.03	1.04	1.10
All passenger cars	1.11	1.13	1.10	1.12	1.06	1.10	1.09	1.12	1.09	1.07	1.04	1.05	1.09
Trucks and truck combinations:													
Single-unit trucks	1.01	.85	.99	.93	1.09	1.10	1.08	1.02	1.07	1.08	.84	.96	1.05
Truck combinations	1.10	1.17	1.31	1.22	1.02	1.11	1.06	1.05	1.04	1.30	1.26	1.27	1.10
All trucks and truck combinations	1.03	.93	1.06	1.01	1.06	1.10	1.07	1.03	1.06	1.12	.95	1.07	1.06
Busses	.75	.99	.99	.95	.87	1.06	.84	1.02	.94	.94	.97	.96	.95
All vehicles	1.09	1.09	1.09	1.09	1.06	1.10	1.08	1.09	1.08	1.08	1.02	1.06	1.08

[1] The ratios for "all vehicles" are based on year-round automatic recorder data, while those for the individual vehicle types are based principally on summer counts.

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[4] Trucks and truck combinations weighing 13 tons or more or having an axle weighing 18,000 pounds or more.

[5] In this article, the term "truck" is used to indicate a single-unit vehicle; "truck combination" to indicate truck-tractor semitrailer (with or without full trailer) and truck with full trailer; and "trucks and truck combinations" to indicate all of these vehicles together.

Table 3.—*Percentage distribution of travel, by vehicle type and by type of operation, on main rural roads in the summer of 1949*

Vehicle type	Eastern regions				Central regions					Western regions			United States average	U. S. percentage distribution of trucks and truck combinations by type of operation	
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average		Private	For hire
Passenger cars:															
Local	56.11	63.31	57.15	59.61	58.56	45.37	59.79	58.84	57.13	41.75	76.14	63.83	59.12	------	------
Foreign	25.38	16.07	19.03	18.66	20.74	22.46	15.41	15.04	18.37	34.24	10.56	19.04	18.58	------	------
All passenger cars	81.49	79.38	76.18	78.27	79.30	67.83	75.20	73.88	75.50	75.99	86.70	82.87	77.70	------	------
Single-unit trucks:															
Panel and pickup	5.31	4.35	6.89	5.60	5.32	12.11	7.05	10.02	7.77	9.76	3.36	5.65	6.69	40.36	1.68
Other 2-axle, 4-tire	.65	1.18	1.04	1.04	.42	.57	1.02	.37	.57	.79	.52	.62	.73	4.38	.33
Other 2-axle, 6-tire	7.16	7.49	7.91	7.63	6.62	11.70	9.79	7.90	8.35	6.99	3.54	4.77	7.49	39.18	22.36
3-axle	.34	.42	.35	.38	.32	.22	.26	.18	.26	.36	.54	.48	.33	1.45	2.03
All single-unit trucks	13.46	13.44	16.19	14.65	12.68	24.60	18.12	18.47	16.95	17.90	7.96	11.52	15.24	85.37	26.40
Truck-tractor and semitrailer combinations:															
3-axle	3.58	5.05	4.89	4.77	4.58	4.85	3.33	4.65	4.34	1.74	.77	1.11	3.92	10.43	45.86
4-axle	.15	1.06	1.29	1.03	2.14	.80	2.09	1.67	1.83	1.34	.85	1.02	1.43	3.02	19.31
5-axle or more	(1)	.01	(1)	.01	.23	.02	.19	.04	.15	.86	1.36	1.19	.28	.40	4.46
All truck-tractor and semitrailer combinations	3.73	6.12	6.18	5.81	6.95	5.67	5.61	6.36	6.32	3.94	2.98	3.32	5.63	13.85	69.63
Truck and trailer combinations:															
4-axle or less	.02	.01	.01	.01	.05	(1)	.20	.17	.11	.23	.22	.22	.10	.39	.65
5-axle	----	----	----	----	.17	----	(1)	.01	.07	.34	.39	.38	.10	.18	1.46
6-axle or more	----	----	----	----	.11	----	(1)	----	.04	.32	.73	.58	.12	.21	1.86
All truck and trailer combinations	.02	.01	.01	.01	.33	(1)	.20	.18	.22	.89	1.34	1.18	.32	.78	3.97
All combinations	3.75	6.13	6.19	5.82	7.28	5.67	5.81	6.54	6.54	4.83	4.32	4.50	5.95	14.63	73.60
All trucks and truck combinations	17.21	19.57	22.38	20.47	19.96	30.27	23.93	25.01	23.49	22.73	12.28	16.02	21.19	100.00	100.00
Busses	1.30	1.05	1.44	1.26	.74	1.90	.87	1.11	1.01	1.28	1.02	1.11	1.11	------	------
All vehicles	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	------	------

[1] Less than 0.005 percent.

In this table all single-unit trucks are divided into classification types based on the axle and tire arrangements, while the truck combinations are classified according to the total number of axles of the combination. The classification of vehicles into these types has been used only in the last three annual surveys. It has several advantages over the old "light, medium, and heavy" grouping, among which are more homogeneous groupings and more positive identification of the types.

The data in table 3 indicate that truck travel in 1949 in proportion to total travel was heaviest in the East South Central region and next heaviest in the West South Central region. Somewhat different figures were found in 1948: the West South Central region then had the highest percentage of truck traffic, followed by the West North Central and South Atlantic regions. The percentage of truck traffic in the South Atlantic, East South Central, West North Central, West South Central, and Mountain regions exceeded 20 percent of the total traffic in both 1948 and 1949. More urbanized areas, such as the New England, Middle Atlantic, and Pacific regions, where total traffic is rather heavy, have the smallest percentage of truck travel. The table indicates that certain types of vehicles are popular in some sections. For instance, the truck and trailer combinations with six or more axles and the truck-tractor and semitrailer combinations with five or more

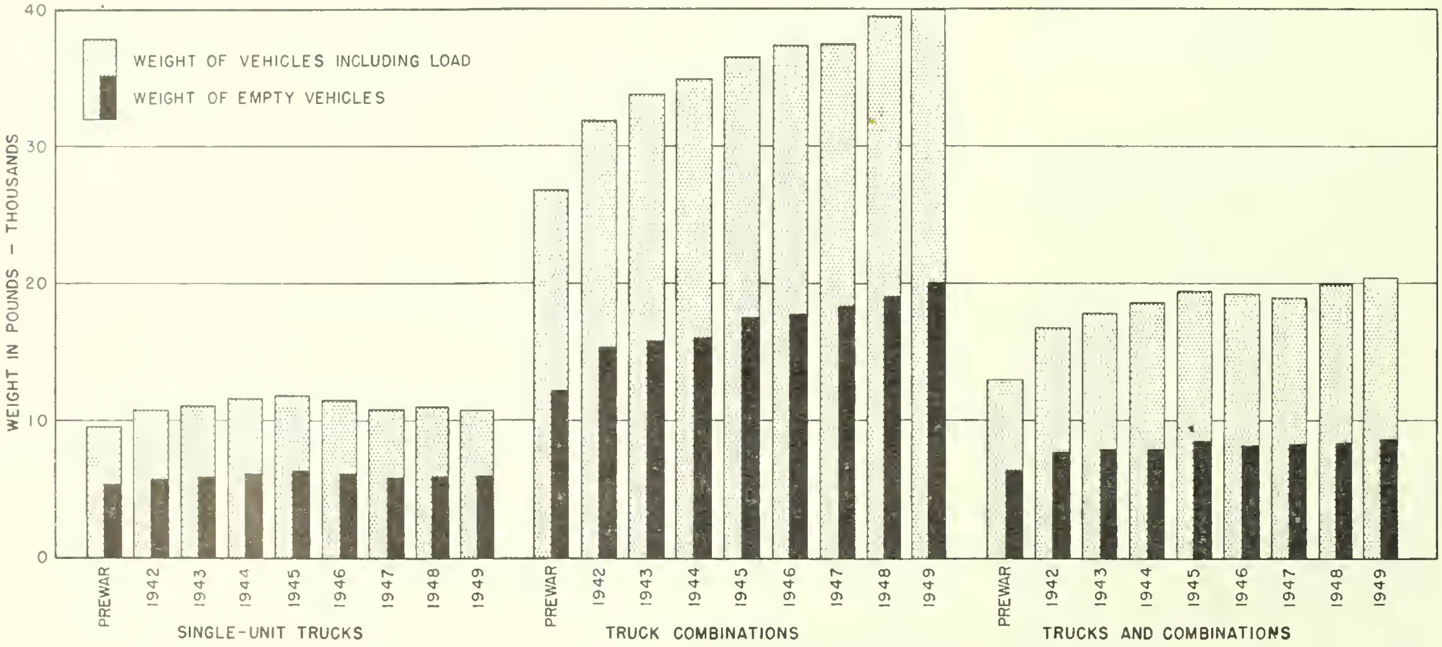

Figure 3.—Average weights of loaded and of empty trucks and truck combinations in the summers of 1942–49 and in a corresponding period of a prewar year.

Table 4.—*Average weights (in pounds) of loaded and empty trucks and truck combinations, by vehicle types, in the summer of 1949*

Vehicle type	Eastern regions				Central regions					Western regions			United States average	U. S. average by type of operation	
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average		Private	For-hire
AVERAGE WEIGHTS OF LOADED VEHICLES															
Single-unit trucks:															
Panel and pickup	4, 901	5, 409	4, 756	5, 015	5, 041	5, 111	5, 154	5, 907	5, 362	5, 695	4, 524	5, 100	5, 242	5, 208	6, 702
Other 2-axle, 4-tire	6, 201	6, 278	6, 301	6, 281	6, 571	6, 974	6, 621	6, 883	6, 677	7, 133	5, 803	6, 302	6, 447	6, 358	10, 221
Other 2-axle, 6-tire	14, 343	15, 159	13, 307	14, 196	13, 007	13, 611	13, 674	12, 959	13, 294	14, 990	12, 262	13, 514	13, 614	13, 233	15, 789
3-axle	30, 595	33, 616	27, 675	30, 716	27, 734	28, 635	25, 951	28, 101	27, 433	26, 728	25, 635	25, 880	28, 128	27, 484	29, 081
Average	11, 352	12, 660	10, 716	11, 565	10, 339	10, 879	10, 920	9, 213	10, 310	11, 486	10, 380	10, 879	10, 765	10, 150	16, 222
Truck combinations:															
Truck-tractor and semitrailer	37, 158	42, 105	37, 898	39, 629	37, 322	34, 364	39, 135	36, 211	37, 100	45, 621	48, 550	47, 405	39, 151	37, 255	40, 173
Truck and trailer	39, 471	60, 366	(1)	48, 506	69, 361	---	18, 109	18, 495	47, 099	62, 642	58, 613	59, 451	55, 458	42, 182	64, 321
Average	37, 174	42, 158	37, 889	39, 653	38, 482	34, 364	38, 462	35, 719	37, 397	48, 009	51, 411	50, 227	39, 999	37, 543	41, 427
Average, all trucks and truck combinations	18, 062	23, 196	19, 152	20, 643	22, 638	17, 422	19, 589	16, 560	19, 774	22, 446	22, 781	22, 643	20, 432	14, 811	35, 374
AVERAGE WEIGHTS OF EMPTY VEHICLES															
Single-unit trucks:															
Panel and pickup	4, 035	4, 383	3, 851	4, 039	3, 971	4, 078	4, 205	4, 446	4, 189	4, 141	3, 824	4, 038	4, 121	4, 099	4, 784
Other 2-axle, 4-tire	5, 074	4, 992	5, 126	5, 048	4, 836	5, 044	5, 271	5, 139	5, 102	5, 285	4, 585	4, 971	5, 057	5, 018	7, 131
Other 2-axle, 6-tire	8, 651	8, 966	7, 310	8, 182	7, 668	7, 728	8, 059	7, 692	7, 784	8, 285	7, 482	7, 978	7, 942	7, 828	8, 636
3-axle	17, 169	15, 230	13, 143	14, 609	14, 500	11, 953	13, 585	16, 187	14, 489	14, 377	14, 139	14, 225	14, 483	14, 519	13, 644
Average	6, 448	6, 895	5, 424	6, 108	5, 885	5, 529	6, 033	5, 991	5, 864	5, 553	5, 572	5, 560	5, 903	5, 724	8, 716
Truck combinations:															
Truck-tractor and semitrailer	20, 672	20, 646	18, 787	19, 815	19, 588	16, 690	20, 250	18, 670	19, 094	23, 016	22, 103	22, 592	19, 610	19, 088	19, 945
Truck and trailer	---	---	(1)	(1)	31, 825	---	12, 393	15, 127	25, 124	29, 595	27, 837	28, 523	27, 142	23, 806	29, 630
Average	20, 672	20, 646	18, 784	19, 814	20, 389	16, 690	19, 906	18, 579	19, 330	24, 722	24, 296	24, 503	20, 019	19, 314	20, 546
Average, all trucks and truck combinations	8, 811	9, 782	7, 694	8, 655	10, 119	6, 781	8, 852	8, 673	8, 700	8, 142	8, 832	8, 406	8, 648	7, 252	16, 484

[1] Data omitted because of insufficient sample.

axles are used more frequently in the Pacific region than in any other area. Combinations involving trailers are used much less in the East South Central region and in the three eastern regions. The percentage of travel by all combinations has increased steadily over that in the previous years' samples, this percentage being 5.95 in 1949, 5.84 in 1948, 5.73 in 1947, and 5.26 in 1946.

Private and For-Hire Traffic

In the survey conducted in 1949 information was gathered in most of the participating States concerning the use-classification under which each vehicle was being operated. Data were reported separately for private and for-hire vehicles of each type, showing the percentage loaded and the average weight of loaded and of empty vehicles. The operation-use classification of each of the heavy vehicles—those with one or more axles weighing 18,000 pounds or more, or with a gross weight of 26,000 pounds or more—generally was designated. This information made possible the calculating of vehicle-mileages, ton-mileages, and other data on the main rural roads by the various types of trucks and truck combinations privately operated and operated for-hire.

In the last two columns of table 3 are shown the percentage distribution of travel of all privately operated and for-hire trucks and combinations, by vehicle type. In general the lighter types of vehicles predominate in the private classification and, conversely, the heavier vehicles constitute a much higher proportion of the for-hire vehicles. This difference is very noticeable in the percentages for the light panel and pickup trucks and for the heavy three-axle truck-tractors with semitrailers. Over 40 percent of the privately operated truck travel was by the panel and pickup type, while less than 2 percent of the for-hire vehicle travel was by this type. On the other hand, less than 15 percent of the travel of all privately operated vehicles was by truck combinations, while of the for-hire travel almost 74 percent was by combinations.

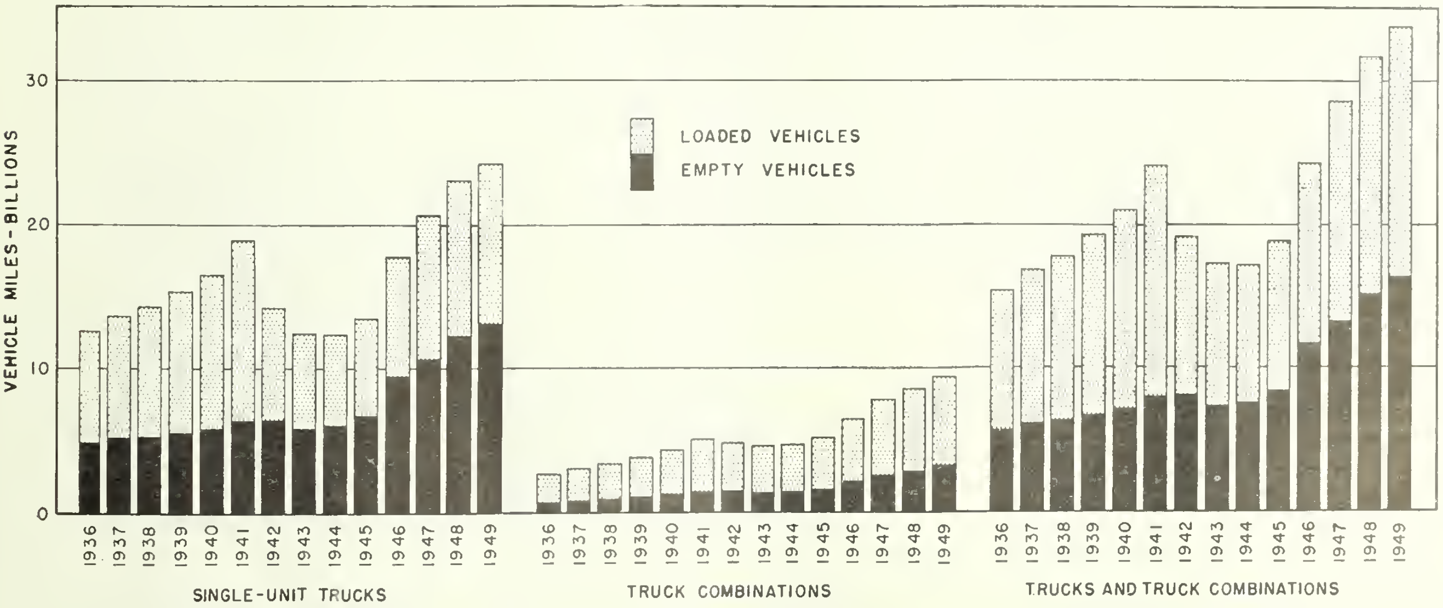

Figure 4.—Travel on main rural roads, 1936–49, by loaded and by empty trucks and truck combinations.

Table 5.—*Comparison of estimated vehicle-miles of travel on main rural roads in 1936, 1941, 1946, 1948, and 1949*

Year	All vehicles, vehicle-miles	Passenger cars and busses [1]		All trucks and truck combinations		Single-unit trucks		Truck combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles
	Millions		*Millions*		*Millions*		*Millions*		*Millions*
1936	88, 412	82. 6	73, 005	17. 4	15, 407	82. 1	12, 650	17. 9	2, 757
1941	122, 505	80. 3	98, 320	19. 7	24, 185	78. 8	19, 057	21. 2	5, 128
1941: 1936 ratio	*1. 39*	*. 97*	*1. 35*	*1. 13*	*1. 57*	*. 96*	*1. 51*	*1. 18*	*1. 86*
1946	124, 149	80. 4	99, 803	19. 6	24, 346	73. 3	17, 838	26. 7	6, 508
1946: 1941 ratio	*1. 01*	*1. 00*	*1. 02*	*. 99*	*1. 01*	*. 93*	*. 94*	*1. 26*	*1. 27*
1946: 1936 ratio	*1. 40*	*. 97*	*1. 37*	*1. 13*	*1. 58*	*. 89*	*1. 41*	*1. 49*	*2. 36*
1948	147, 597	78. 5	115, 837	21. 5	31, 760	72. 9	23, 138	27. 1	8, 622
1949	159, 379	78. 8	125, 602	21. 2	33, 777	71. 9	24, 295	28. 1	9, 482
1949: 1948 ratio	*1. 08*	*1. 00*	*1. 08*	*. 99*	*1. 06*	*. 99*	*1. 05*	*1. 04*	*1. 10*
1949: 1941 ratio	*1. 30*	*. 98*	*1. 28*	*1. 08*	*1. 40*	*. 91*	*1. 27*	*1. 33*	*1. 85*
1949: 1936 ratio	*1. 80*	*. 95*	*1. 72*	*1. 22*	*2. 19*	*. 88*	*1. 92*	*1. 57*	*3. 44*
COMPARISON FOR TRUCKS AND TRUCK COMBINATIONS, BY TYPE OF OPERATION									
Private:									
1936	------	------	------	78. 8	12, 140	86. 7	10, 963	42. 7	1, 177
1949	------	------	------	77. 2	26, 077	91. 6	22, 262	40. 2	3, 815
1949: 1936 ratio	------	------	------	*. 98*	*2. 15*	*1. 06*	*2. 03*	*. 94*	*3. 24*
For-hire:									
1936	------	------	------	21. 2	3, 267	13. 3	1, 687	57. 3	1, 580
1949	------	------	------	22. 8	7, 700	8. 4	2, 033	59. 8	5, 667
1949: 1936 ratio	------	------	------	*1. 08*	*2. 36*	*. 63*	*1. 21*	*1. 04*	*3. 59*

[1] Percentages of total 1949 travel by passenger cars and by busses are reported separately in table 3

Average Weights Increase

The average weights of loaded and empty trucks and truck combinations, separately and combined, are shown graphically in figure 3 for each year from 1942 to 1949, inclusive, and in a prewar year, generally 1936 or 1937. The weights of single-unit trucks, both loaded and empty, increased each year from the 1936–37 period through 1945, then leveled off or decreased slightly each succeeding year. At the same time weights of truck combinations, both loaded and empty, have increased each year during the period shown. The increase in average weight of loaded combinations from the 1936–37 period to 1949 was almost 50 percent, compared to only 12 percent for single-unit trucks. The increase for all trucks and truck combinations combined was 57 percent, a figure higher than that of either type separately, because of the increased proportion of combinations in the latter years.

The average weights of the various types of loaded and empty trucks and truck combinations in the summer of 1949 are shown in table 4 for the different regions. This table brings out clearly the important differences that exist in the weight characteristics of the vehicles in the different groups. It will be noted, for example, that for the United States as a whole, the loaded three-axle, single-unit trucks weighed about twice as much as the two-axle, six-tire trucks which, in turn, weighed about twice as much as the two-axle, four-tire trucks. Similar differences existed throughout the various classifications. On the other hand the regional differences in average weight for each of the vehicle types that are common throughout the country are surprisingly small. The extremely low weights of truck and trailer combinations in the West North Central and West South Central regions indicate a predominance of small, home-made trailers of low capacity.

The average weights of loaded and empty trucks and truck combinations operated privately and for-hire in the summer of 1949 are shown in the last two columns of table 4. The for-hire vehicles, when compared by types, are generally heavier than those operated privately, and the average of all types of for-hire vehicles, either loaded or empty, is more than twice as heavy as the average of the privately operated vehicles. It was shown in table 3 that the largest portion of the private vehicles consisted of the small single-unit trucks while the greater portion of the for-hire vehicles consisted of the heavy truck combinations. This decided difference in the distributions of sizes of vehicles in the two operation classes accounts for the great difference between their average weights.

Truck Travel Again Increases

Figure 4 shows a comparison of the estimated vehicle-mileage of travel by loaded and empty single-unit trucks and truck combinations, separately and combined, on main rural roads, for each year from 1936 to 1949, inclusive. This chart demonstrates graphically the steady growth of truck traffic during the prewar years, 1936–41, the temporary effect of wartime restrictions in the period 1942–45, and the remarkable increases in truck transportation that have occurred since the end of hostilities in 1945.

Table 5 shows a comparison of the estimated vehicle-mileage of travel by vehicles of different types on all main rural roads in 1936, the earliest year for which comprehensive weight data are available; in 1941, the peak prewar year, 5 years after the beginning of the surveys; in 1946, 10 years after the beginning of the surveys; and in 1948 and

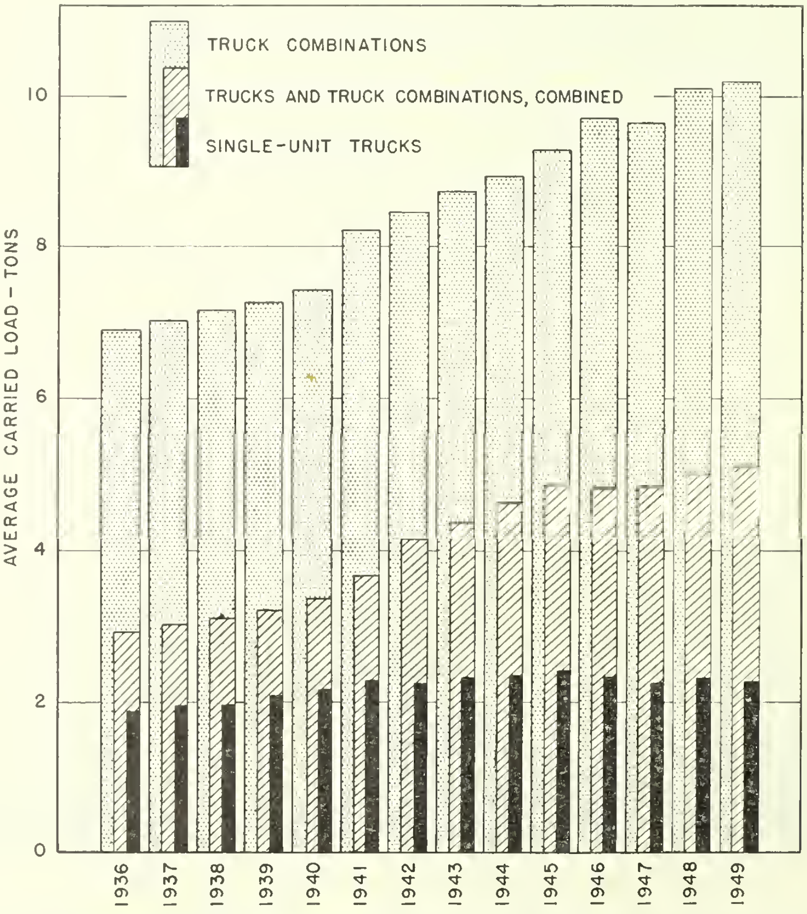

Figure 5.—Average load carried by trucks and truck combinations on main rural roads, 1936–49.

1949. The ratios of 1949 travel to that of the preceding years indicate that increases for trucks and truck combinations generally were greater than for passenger cars, and that increases for truck combinations were greater than for single-unit trucks. In the 13 years from 1936 to 1949, passenger-car and bus travel combined increased over 70 percent, travel by all trucks and combinations more than doubled, and travel by truck combinations (considered separately) more than tripled.

The lower portion of table 5 shows a comparison of the estimated vehicle-mileage of travel in 1936 and in 1949 by privately operated trucks and combinations, and by those operated for-hire. Travel by for-hire vehicles increased somewhat more than travel by private vehicles, the 1949:1936 ratio being 2.36 in the case of for-hire vehicles and 2.15 in the case of private vehicles. Most of the increase in for-hire vehicle travel was by truck combinations, there being only a 21-percent increase in the for-hire vehicle-mileage by single-unit trucks compared to a 259-percent increase by combinations. In the case of the private vehicles, on the other hand, there were substantial increases in the vehicle-mileage by both types.

In the lower portion of table 5, incidentally, the percentage figures refer to trucks and truck combinations only; for example, of all single-unit trucks, in 1936, 86.7 percent were in private operation and 13.3 percent for-hire.

Volume of Highway Freight

Figure 5 gives a comparison of the average load carried by single-unit trucks and truck combinations, separately and combined, in the 14 years that the planning surveys have been operating. The general trend of load weights was upward throughout the period. The slight decline in the weights of loads carried by single-unit trucks since 1945 has been more than offset by the increased use of combinations and the increased weights of loads carried by vehicles of this type.

Figure 6 shows a comparison for each year from 1936 through 1949 of the ton-mileage of freight carried by trucks and truck combinations on main rural roads. The chart demonstrates clearly that truck combinations are transporting each year a larger proportion of the total amount of highway freight. In 1936 the truck combinations hauled slightly less ton-mileage than the single-unit trucks, while in 1949 they hauled almost two and one-half times as much. The rapid rate of annual increase in total freight carried, which took place in the years immediately following the war, has been reduced in the last two years to a rate of increase more nearly comparable with that of prewar years.

In table 6 is shown a comparison of the percentage of vehicles carrying loads, the average carried load, and the ton-mileage carried for all trucks and combinations, for single-unit trucks, and for truck combinations in 1949 and the other significant periods used in table 5. The trend from 1936 to 1949 of average weight carried, shown graphically in figure 5, and that of the ton-mileage transported during the same period, shown in figure 6, have already been discussed.

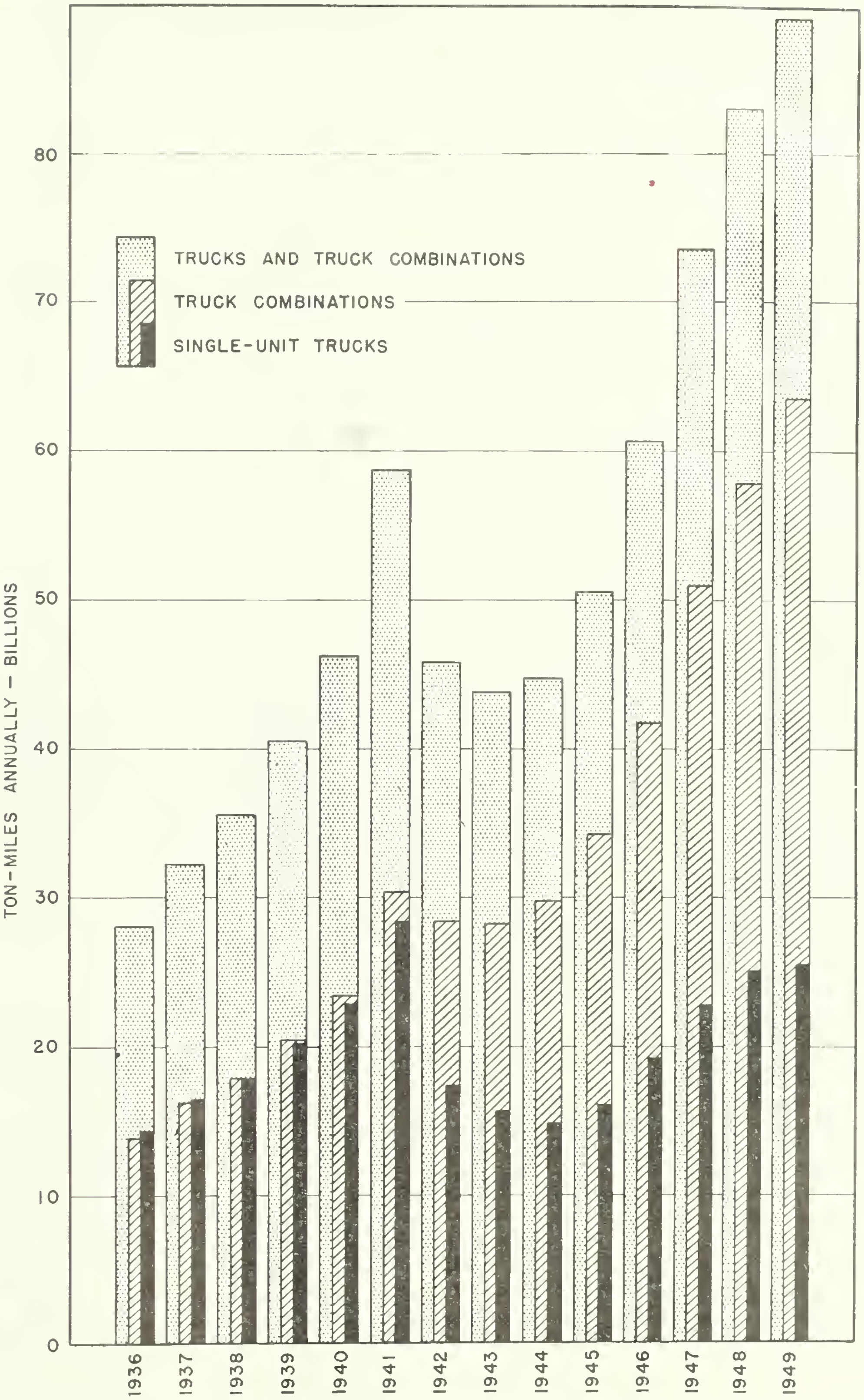

Figure 6.—Ton-miles carried by trucks and truck combinations on main rural roads, 1936–49.

In the country as a whole, the percentage of trucks and truck combinations carrying loads decreased slightly from 1948 to 1949. The percentage loaded for single-unit trucks, for truck combinations, and for the two types of vehicles combined was less than the comparable figures for 1946, when the downward trend in these figures that had been maintained during the war appeared to be halted temporarily. Since 1946 the trend has not been clearly defined, for these percentages increased slightly in 1947 and then decreased in 1948 and again in 1949. With these latest decreases the proportion of vehicles loaded

Table 6.—*Comparison of the estimated percentage of trucks and truck combinations loaded, average carried load, and ton-miles carried on main rural roads in 1936, 1941, 1946, 1948, and 1949, and similar data for privately operated and for-hire vehicles*

Year	All trucks and truck combinations			Single-unit trucks			Truck combinations		
	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried
		Tons	*Millions*		*Tons*	*Millions*		*Tons*	*Millions*
1936	62.8	2.90	28,005	60.7	1.86	14,258	72.2	6.90	13,747
1941	66.7	3.64	58,737	65.4	2.29	28,487	71.6	8.23	30,250
1941:1936 ratio	*1.06*	*1.26*	*2.10*	*1.08*	*1.23*	*2.00*	*.99*	*1.19*	*2.20*
1946	51.7	4.84	60,892	46.4	2.31	19,101	66.2	9.70	41,791
1946:1941 ratio	*.78*	*1.33*	*1.04*	*.71*	*1.01*	*.67*	*.92*	*1.18*	*1.38*
1946:1936 ratio	*.82*	*1.67*	*2.17*	*.76*	*1.24*	*1.34*	*.92*	*1.41*	*3.04*
1948	52.2	5.02	83,119	46.8	2.33	25,219	66.5	10.10	57,900
1949	51.6	5.11	89,100	46.1	2.29	25,639	65.7	10.19	63,461
1949:1948 ratio	*.99*	*1.02*	*1.07*	*.99*	*.98*	*1.02*	*.99*	*1.01*	*1.10*
1949:1941 ratio	*.77*	*1.40*	*1.52*	*.70*	*1.00*	*.90*	*.92*	*1.24*	*2.10*
1949:1936 ratio	*.82*	*1.76*	*3.18*	*.76*	*1.23*	*1.80*	*.91*	*1.48*	*4.62*
Privately operated trucks and truck combinations:									
1936	60.3	2.20	16,094	59.8	1.71	11,180	65.5	6.37	4,914
1949	47.6	3.48	43,231	45.3	2.10	21,193	61.2	9.43	22,038
1949:1936 ratio	*.79*	*1.58*	*2.66*	*.76*	*1.23*	*1.90*	*.93*	*1.48*	*4.48*
For-hire trucks and truck combinations:									
1936	71.9	5.07	11,911	66.4	2.73	3,078	77.3	7.23	8,833
1949	65.1	9.16	45,869	55.1	3.97	4,446	68.7	10.65	41,423
1949:1936 ratio	*.91*	*1.81*	*3.85*	*.83*	*1.45*	*1.44*	*.89*	*1.47*	*5.03*

reached a new all-time low level, only 46 percent of the single-unit trucks and less than 66 percent of the truck combinations being loaded in 1949, compared to 61 percent and 72 percent, respectively, for these two types of vehicles in 1936.

The lower portion of table 6 shows a comparison of the percentage loaded, average carried load, and ton-mileage for single-unit trucks, truck combinations, and these two types of vehicles combined, when operated as private and as for-hire vehicles. An appreciably larger percentage of the for-hire vehicles are loaded; the loads carried by these vehicles are much heavier; and the average carried loads are increasing at a more rapid rate than for privately operated vehicles. Single-unit trucks transport an important part of the freight moved in privately operated vehicles, but only a minor part of the freight moved by for-hire trucks.

The first part of table 7 gives a detailed comparison of the percentage of vehicle-miles of travel, percentage of vehicles loaded, average carried load, and percentage of total ton-mileage carried by the various types of trucks and truck combinations traveling on main rural roads in 1948 and 1949. Many interesting comparisons can be made from this table showing the relative importance from a freight-carrying standpoint of different portions of the traffic stream. In 1949, for instance, while panel and pickup trucks traveled almost 32 percent of the vehicle-mileage, they accounted for less than 3 percent of the ton-mileage. The truck-tractor and semitrailer combinations traveled less than 27 percent of the vehicle-mileage, but they carried almost 66 percent of the ton-mileage.

From the columns in table 7 showing the percentage loaded, by types, it can be observed that the percentage of vehicles carrying loads increases directly as the size of the vehicle type, extending from the light panel and pickup trucks that are loaded 36 percent of the time, to the heavy combinations that are loaded over 65 percent of the time.

The right-hand portion of table 7 shows the percentage loaded, average carried load, and percentage of total ton-mileage carried by various types of privately operated trucks and truck combinations compared to those operated for-hire on the main rural roads in 1949. The percentage of travel (vehicle-mileage) by these types is given in table 3. A comparison of vehicle-mileage percentage with ton-mileage percentage, by operating classes, shows that panel and pickup trucks, privately operated, travel over 40 percent of the vehicle-mileage while transporting only about 5 percent of the freight moved in privately operated vehicles. At the same time, for-hire panels and pickups travel less than 2 percent of the total for-hire vehicle-mileage and carry only 0.2 percent of the total ton-mileage moved by the for-hire vehicles. The heavy-vehicle combinations, privately operated, travel about 15 percent of the total mileage and carry almost 51 percent of the freight moved by privately operated vehicles, while the for-hire combinations travel almost 74 percent of the total vehicle-mileage of all such vehicles and carry over 90 percent of the freight transported by all the for-hire trucks and combinations.

Gross Weights Increasing Slightly

Figure 7 shows for the United States as a whole the frequency of gross weights by years, from the prewar years (generally 1936 or 1937) to 1949, of 30,000 pounds or more, of 40,000 pounds or more, and of 50,000 pounds or more. The trend of frequency of heavy loads continues upward although the frequency of 50,000-pound weights was slightly less in 1949 than in the previous year. The frequency of these heaviest loads was 12 times greater in 1949 than in the prewar year, the weights of 40,000 pounds or more were 7 times as frequent, and those of 30,000 pounds or more were 3½ times as frequent as in the earliest years of the surveys. Vehicles weighing 30,000 pounds or more and 40,000 pounds or more appear in greater numbers than ever before, whereas the number of those in the heaviest group declined slightly.

The 1949 gross-weight frequency data by vehicle type and region are presented in table 8. No panels, pickups, or other two-axle, four-tire, single-unit trucks were found in the survey weighing as much as 30,000 pounds, so there is no entry for these vehicles in the table, though they are included in the total number of vehicles weighed in computing the frequencies for all trucks and combinations. Heavy gross weights are much more frequent in the Pacific region than in other parts of the country. In this region 99 of each 1,000 trucks and truck combinations on the main rural highways in 1949, empties included, weighed 50,000 pounds or more. This fre-

Table 7.—*Percentage of vehicle-miles of travel, percentage loaded, average carried load, and percentage of total ton-miles carried by various types of trucks and truck combinations on main rural roads in 1949 compared to that in corresponding months of 1948*

Vehicle type	Percentage of vehicle-miles of travel		Percentage loaded		Average carried load		Percentage of ton-miles carried		Distribution by type of operation in 1949					
									Percentage loaded		Average carried load		Percentage of ton-miles carried	
	1949	1948	1949	1948	1949	1948	1949	1948	Private	For-hire	Private	For-hire	Private	For-hire
Single-unit trucks:					*Tons*	*Tons*					*Tons*	*Tons*		
Panel and pickup	31.55	28.90	35.9	36.7	0.64	0.64	2.75	2.60	35.72	48.13	0.63	1.42	5.45	0.19
Other 2-axle, 4 tire	3.46	3.59	49.4	51.9	.78	1.24	.50	.88	49.39	47.75	.74	2.76	.96	.07
Other 2-axle, 6-tire	35.34	38.66	54.5	53.6	3.17	3.10	23.15	24.51	54.40	55.40	3.06	3.82	39.28	7.94
3-axle	1.58	1.70	54.8	55.2	7.23	6.57	2.38	2.35	53.44	58.09	7.11	7.52	3.33	1.49
All single-unit trucks	71.93	72.85	46.1	46.8	2.29	2.33	28.78	30.34	45.30	55.05	2.10	3.97	49.02	9.69
Truck combinations:														
Truck-tractor and semitrailer	26.57	25.35	65.8	66.2	9.95	9.83	65.91	63.09	61.11	68.98	9.39	10.27	47.98	82.82
Truck and trailer	1.50	1.80	63.4	70.2	14.69	13.64	5.31	6.57	63.60	63.24	10.08	17.75	3.00	7.49
All truck combinations	28.07	27.15	65.7	66.5	10.19	10.10	71.22	69.66	61.24	68.67	9.43	10.65	50.98	90.31
All trucks and truck combinations	100.00	100.00	51.6	52.2	5.11	5.02	100.00	100.00	47.63	65.07	3.48	9.16	100.00	100.00

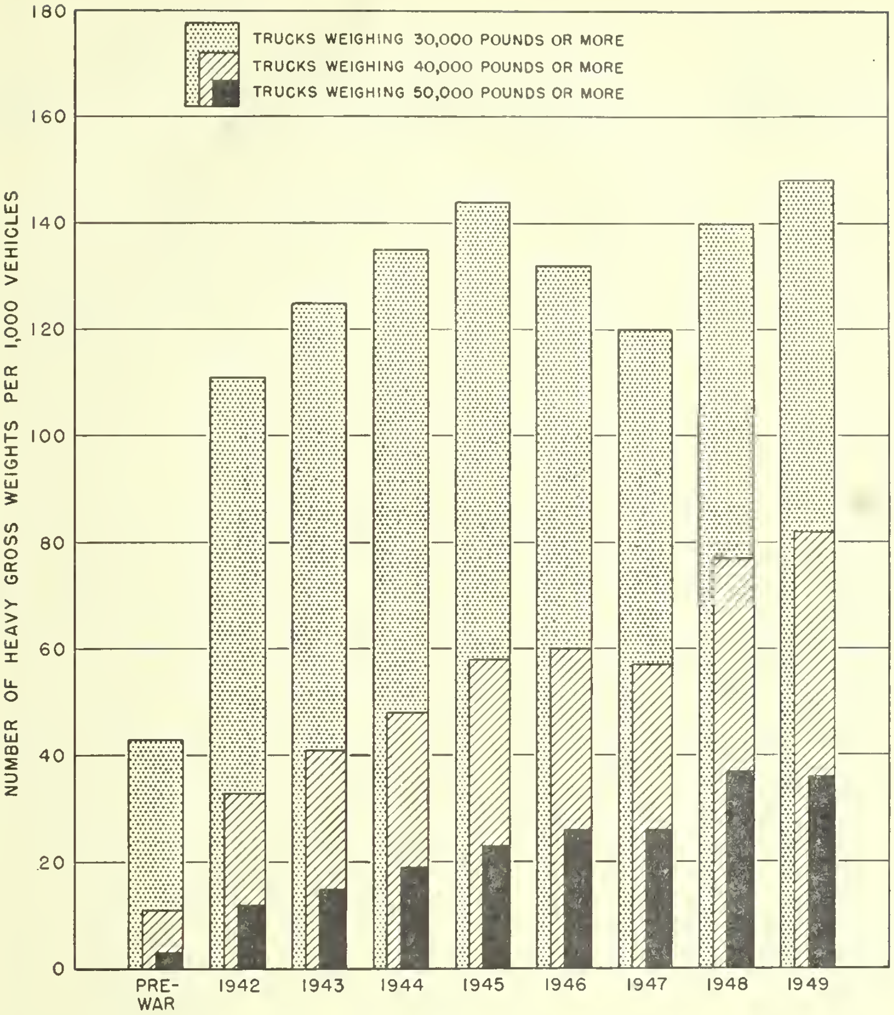

Figure 7.—Number of heavy gross weights per 1,000 trucks and truck combinations (empties included) in the summers of 1942–49 and a prewar year.

quency is the same as was found in 1948 but is slightly lower than the 1946 figure of 104 for each 1,000 in this heavy category. These heavy vehicles are almost entirely of the combination type, approximately 40 percent of all combinations observed, or about 52 percent of the loaded ones, weighing 50,000 pounds or more. In the East South Central region these heavy gross weights were about one-fifteenth as frequent as in the Pacific region, while in the New England, South Atlantic, and West South Central regions the heavy weights were one-fifth as frequent as in the Pacific region.

Frequency of Heavy Axle Loads

Figure 8 shows the frequency of axle loads of 18,000 pounds or more, of 20,000 pounds or more, and of 22,000 pounds or more, by years from the prewar years (1936–37) to 1949. The frequency of heavy axle loads increased year by year from the prewar period through 1948. The frequencies for 1949 are slightly lower than those found in the previous year, yet are higher than in any other previous period and fit very closely the long-time trend from 1942 through 1947. Axle loads in excess of 22,000 pounds increased in frequency from 2 per 1,000 vehicles in the prewar period to 17 per 1,000 vehicles in 1949, an increase of 750 percent. Somewhat lesser increases occurred in the frequencies of axle loads from 18,000 to 20,000 pounds. The decline in heavy axle loads in all the categories from 1948 to 1949, coupled with the decline in frequency of the heaviest gross loads shown in figure 7, may possibly indicate that increased enforcement activity is being reflected in better law observance. One year's reversal in trend is insufficient to warrant conclusions. The 1950 survey results should show whether the trend will continue downward from 1948 or resume the steady upward course followed prior to that year.

Table 9 gives data concerning the number of heavy axle loads per 1,000 loaded and empty trucks and truck combinations of various types on the main rural roads in the summer of 1949. Since no panel or pickup trucks were found with axles weighing 18,000 pounds or more, there is no entry for these in the table, though they are included in figuring the frequencies for all trucks and truck combinations.

Though the greatest frequency of heavy gross weights is in the Pacific region, as shown in table 8, the lowest frequency of heavy axle loads is in this same region. The greatest frequency is in the Middle Atlantic region and the next greatest in New England. In these two regions the relatively high frequency is attributable mainly to the large number of two-axle truck-tractors pulling one-axle or two-axle semitrailers. The relative infrequency of heavy axles in the Pacific region, in the presence of a large proportion of heavy gross loads, indicates a better distribution of the loads over a larger number of axles.

Loads Above Legal Limits

Table 10[6] shows the number of trucks and truck combinations of each type, per 1,000 such vehicles counted, empties included, that exceeded the permissible axle, axle-group, or gross-weight legal limits in effect in the individual States in the summer of 1949, and the number per 1,000 that exceeded these limits by various percentages. Loads in excess of the State legal limits were most frequent, as in 1948, in the East North Central region where it was found that 63 of each 1,000 vehicles exceeded a State weight limit. The Middle Atlantic region stood second in frequency of violations, with 59 vehicles out of each 1,000 exceeding a load limit, followed closely by the Mountain region with 58 violations per 1,000 vehicles.

A comparison of the frequency of loads exceeding State legal limits in 1949, shown in table 10, with similar data collected in the previous year, indicates that these excessive loads have decreased in all areas except the Mountain and Pacific regions. In the Mountain region the frequency increased 45 percent, while in the Pacific region the increase was about 9 percent. For the country as a whole, excessive loads in 1949 were slightly less frequent than in 1948 but slightly more than in 1947. No panel or pickup truck was weighed that exceeded any of the State weight regulations and this classification is omitted from the tables although the number of such vehicles counted is included in the calculations.

Recommended Weight Limits

Uniform regulations concerning maximum allowable gross weights, axle weights, and axle-group weights have been adopted as a policy by the American Association of State Highway Officials and recommended to the State governments for adoption.[7] This policy recommends that no axle shall carry a load in excess of 18,000 pounds and no group of axles shall carry a load in excess of amounts specified in a table of permissible weights based on the distance between the extremes of any group of axles.

In table 11 is shown the number of axles per 1,000 vehicles of various types that exceeded the axle-load limit of 18,000 pounds recommended by the A.A.S.H.O. and the number exceeding these limits by various percentages.

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[6] Tables 10–14 are on pages 96–98.

[7] *Policy concerning maximum dimensions, weights, and speeds of motor vehicles to be operated over the highways of the United States*, adopted April 1, 1946, by the American Association of State Highway Officials; published by the Association in 1946.

Table 8.—*Heavy gross weights per 1,000 loaded and empty trucks and truck combinations on main rural roads, summer of 1949*

Vehicle type [1]	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
NUMBER PER 1,000 WEIGHING 30,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	16	19	3	11	(2)	(2)	(2)	2	1	6	(2)	4	5
3-axle	286	371	245	309	221	298	212	231	229	206	208	207	253
Average	16	22	8	14	6	3	3	3	4	7	14	10	8
Truck combinations:													
Truck-tractor and semitrailer	475	595	498	539	521	458	517	434	492	569	644	612	520
Truck and trailer	(3)	(3)	(3)	--------	789	0	82	53	495	783	722	739	657
Average	476	596	498	540	534	457	502	424	492	613	668	647	528
Average, all trucks and truck combinations	117	191	130	153	208	87	139	107	144	118	176	147	148
NUMBER PER 1,000 WEIGHING 40,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	1	1	0	(2)	0	0	(2)	0	(2)	(2)	0	(2)	(2)
3-axle	137	157	40	107	34	0	14	112	38	69	12	28	61
Average	4	5	1	3	1	0	(2)	1	1	1	1	1	1
Truck combinations:													
Truck-tractor and semitrailer	287	390	285	332	262	189	293	221	250	380	481	438	297
Truck and trailer	(3)	(3)	(3)	--------	512	0	67	31	323	461	523	506	448
Average	289	391	285	333	274	189	285	215	252	397	494	456	305
Average, all trucks and truck combinations	66	120	71	90	105	36	77	54	73	75	121	97	82
NUMBER PER 1,000 WEIGHING 50,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	0	0	0	0	0	0	0	0	0	(2)	0	(2)	(2)
3-axle	6	15	4	9	21	0	0	0	10	14	2	5	9
Average	(2)	1	(2)	(2)	1	0	0	0	(2)	(2)	(2)	(2)	(2)
Truck combinations:													
Truck-tractor and semitrailer	69	170	87	122	109	28	124	75	94	243	382	323	127
Truck and trailer	0	(3)	(3)	--------	455	0	31	10	278	381	470	446	392
Average	69	171	87	123	125	28	120	74	100	271	409	356	141
Average, all trucks and truck combinations	15	52	21	33	48	6	32	18	29	51	99	75	36

[1] No two-axle, four-tire trucks weighing as much as 30,000 pounds were reported.

[2] Less than 5 per 10,000.

[3] Data omitted because of insufficient sample.

Table 9.—*Heavy axle loads per 1,000 loaded and empty trucks and truck combinations on main rural roads in the summer of 1949*

Vehicle type [1]	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
NUMBER PER 1,000 WEIGHING 18,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 4-tire	0	2	2	2	0	0	0	0	0	0	0	0	1
2-axle, 6-tire	46	58	23	40	10	18	10	11	12	34	14	24	23
3-axle	116	169	55	116	67	33	11	56	48	71	4	23	67
Average	27	38	13	24	7	9	5	5	6	15	7	11	13
Truck combinations:													
Truck-tractor and semitrailer	471	566	361	462	206	238	179	198	203	252	147	192	290
Truck and trailer	0	(2)	0	--------	578	0	50	10	355	188	106	128	210
Average	468	567	361	462	223	237	175	193	208	239	135	175	286
Average, all trucks and truck combinations	124	195	99	140	89	50	50	51	63	57	37	48	86
NUMBER PER 1,000 WEIGHING 20,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 4-tire	0	0	0	0	0	0	0	0	0	0	0	0	0
2-axle, 6-tire	27	36	9	23	4	6	2	3	3	18	3	10	11
3-axle	40	69	27	48	32	0	0	0	15	58	0	16	28
Average	15	22	5	13	3	3	1	1	2	8	1	5	6
Truck combinations:													
Truck-tractor and semitrailer	281	341	172	257	60	86	42	72	62	113	28	64	129
Truck and trailer	0	(2)	0	--------	211	0	14	0	128	44	8	18	61
Average	280	342	172	257	67	86	41	70	64	99	22	52	125
Average, all trucks and truck combinations	73	118	46	78	27	18	12	18	20	26	6	16	38
NUMBER PER 1,000 WEIGHING 22,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 4-tire	0	0	0	0	0	0	0	0	0	0	0	0	0
2-axle, 6-tire	16	17	4	11	1	2	1	1	1	9	1	5	5
3-axle	0	29	8	17	29	0	0	0	14	24	0	7	13
Average	9	11	2	6	1	1	(3)	(3)	1	4	(3)	2	3
Truck combinations:													
Truck-tractor and semitrailer	120	193	66	127	19	22	8	24	18	46	5	22	56
Truck and trailer	0	0	0	0	91	0	12	0	57	23	4	9	25
Average	120	192	66	127	22	22	8	23	20	41	5	19	54
Average, all trucks and truck combinations	33	65	18	39	9	5	3	6	6	11	2	6	17

[1] No panel or pick-up trucks with an 18,000-pound axle load were reported.

[2] Data omitted because of insufficient sample.

[3] Less than 5 per 10,000.

This table emphasizes again the high frequency of heavy axle loads in the Middle Atlantic and New England regions. The number of axles per 1,000 vehicles weighing more than the recommended limits was 175 in the Middle Atlantic and 109 in the New England regions, while only 31 such axles for each 1,000 vehicles were found in the Pacific region. For the United States as a whole axle loads heavier than 18,000 pounds were about 12 percent less frequent in 1949 than in 1948, although almost 14 percent more frequent than in 1947.

Table 12 shows the number of vehicles of various types per 1,000 with an axle-group load in excess of the limits recommended by the A.A.S.H.O. and in excess of these limits by various percentages. For the country as a whole, of each 1,000 loaded and empty trucks and truck combinations, 28 had axle-group loads [8] weighing in excess of the recommended limits, 7 of which exceeded the limits by more than 20 percent. Of each 1,000 combinations weighed, 98 had axle-group loads weighing more than the recommended limits, of which 24 exceeded the limits by more than 20 percent. The frequency of the excessive axle-group loads in 1949 was about 7 percent less than in 1948 and 12 percent above the 1947 frequency.

As might be expected, many vehicles were so loaded that they exceeded more than one recommended weight limit, and some vehicles had more than one axle loaded in excess of the recommended limit. Counting each vehicle only once, regardless of the number of ways in which it exceeded any of the A.A.S.H.O. recommended limits, table 13 was derived to show the number of vehicles per 1,000 of each type, both loaded and empty, that exceeded the limits by various percentages. Those vehicles which exceeded more than one provision of the recommended restrictions were tabulated in the column showing the highest percentage excess only of any item.

In the United States as a whole, 68 vehicles out of every 1,000 were overloaded to some degree, and 19 out of every 1,000 exceeded some one of the provisions by more than 20 percent. The frequency of vehicles exceeding the recommendations by any amount in 1949 was 7 percent less than in 1948 and about 15 percent greater than in 1947.

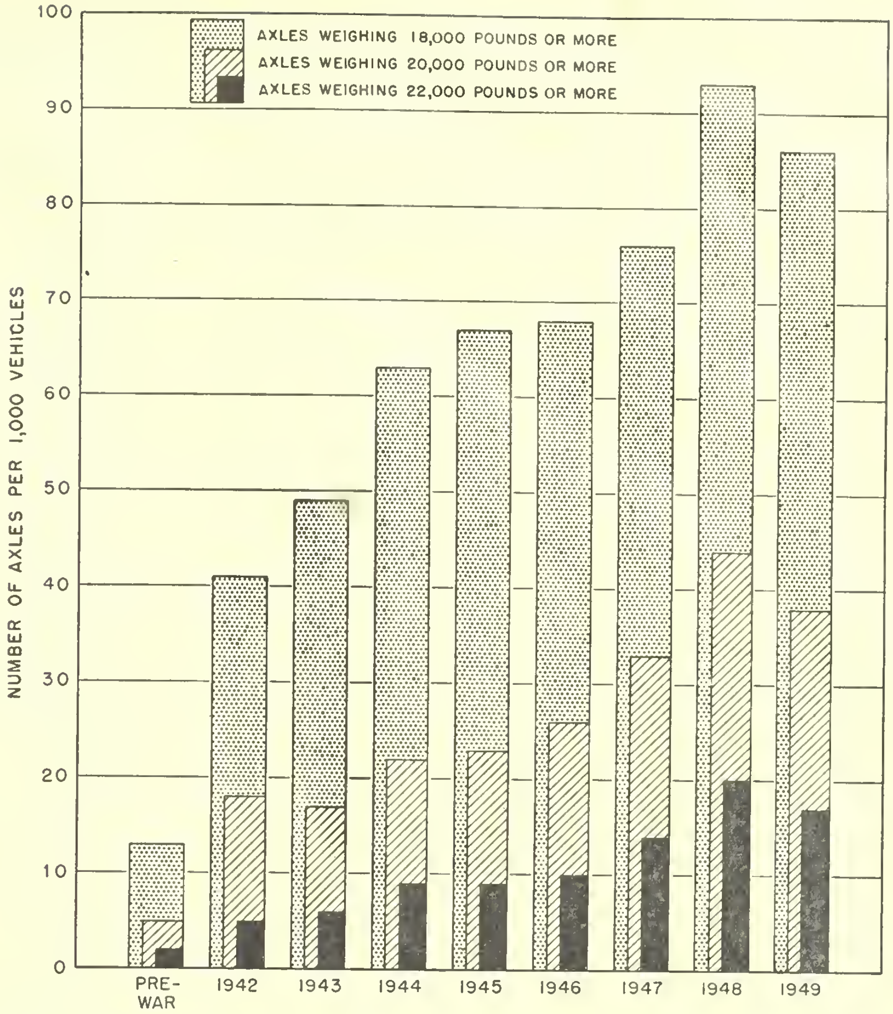

Figure 8.—Number of heavy axle loads per 1,000 trucks and truck combinations (empties included) in the summers of 1942–49 and a prewar year.

In considering the data concerning the frequencies of axles or vehicles exceeding the State legal limits and the A.A.S.H.O. recommendations, especially the frequencies in the Middle Atlantic and New England regions, the fact should be recognized that higher limits generally are permissible under the State laws in this area than are recommended by the Association. Axles exceeding the recommended limits by 25 percent may be within the legal limits of certain States, particularly in these two regions. Some States have no axle limits or axle-group limits in their motor-vehicle restrictions, a fact that complicates direct comparison of excess weights based on law and those based on the recommendations. Comparison of the frequency data given in table 13 with those in table 10 shows that from one-third to one-half of the vehicles exceeding one or more of the Association recommendations actually exceeded a State legal limit.

For-Hire Vehicles More Frequently Overloaded

The first part of table 14 shows the number of privately operated trucks and truck combinations and those operated for-hire, for each 1,000 such loaded and empty vehicles, on main rural roads of the United States, that exceeded some State legal weight limit in 1949. A comparison of the frequency of the excessively loaded vehicles in the two operation classifications shows, in a striking manner, that type by type, the for-hire vehicles generally are more frequently overloaded than are the privately operated ones. For instance, 6 of each 1,000 private single-unit trucks exceeded a State weight limit while 27 of each 1,000 for-hire trucks exceeded the same limits. Likewise, 146 of each 1,000 private truck combinations exceeded State weight limits, while 168 of each 1,000 for-hire combinations exceeded the same limits.

Of each 1,000 vehicles, the frequencies of all private and all for-hire trucks and truck combinations exceeding the State limits were 26 and 131, respectively. In other words, there were five times as many excess loads among the for-hire vehicles as among the privately operated ones. This is as would be expected, because the heavier vehicles are operated more frequently for-hire and the lighter ones for private transportation.

The following parts of table 14 show frequencies of private and for-hire trucks and truck combinations exceeding the A.A.S.H.O. recommended limits for axle loads, for maximum axle-group loads, or for any of the recommended maximum loads. This table shows, in general, that the relation of the frequency of overload of privately operated and of for-hire vehicles is the same when based on A.A.S.H.O. recommendations or on State legal limits.

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[8] The frequency of 28 vehicles of each 1,000 weighed that exceeded the A.A.S.H.O. axle-group recommendation, found in the final analysis of these data as shown in table 12, is slightly larger than the frequency of 2.56 percent (or 26 per 1,000) found in the preliminary analysis reported to the Subcommittee of the Senate Committee on Interstate and Foreign Commerce.

Table 10.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle, axle-group, or gross-weight legal limits in effect in the States by various percentages (maximum) of overload, summer of 1949*

Region and type of vehicle (panel and pickup trucks excluded)	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	15	12	7	4	1	(1)
3-axle	85	76	57	11	6	---
Average, single-unit trucks	10	8	5	2	1	(1)
Truck-tractor and semitrailer	91	53	34	12	5	1
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	91	53	34	12	5	1
Average, all trucks and combinations	28	18	11	4	2	(1)
Middle Atlantic:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	21	13	9	3	1	(1)
3-axle	209	178	115	66	28	---
Average, single-unit trucks	18	13	9	4	1	(1)
Truck-tractor and semitrailer	147	102	73	38	24	6
Truck and trailer	(2)	(2)	(2)	---	---	---
Average, truck combinations	149	104	75	38	24	6
Average, all trucks and combinations	59	42	30	15	8	2
South Atlantic:						
2-axle, 4-tire	1	1	1	1	---	---
2-axle, 6-tire	14	10	7	2	1	(1)
3-axle	41	27	11	8	---	---
Average, single-unit trucks	8	6	4	1	(1)	(1)
Truck-tractor and semitrailer	169	110	70	24	11	1
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	169	110	70	24	11	1
Average, all trucks and combinations	53	35	22	7	7	(1)
East North Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	7	5	3	1	(1)	---
3-axle	72	69	66	43	29	20
Average, single-unit trucks	5	4	3	2	1	(1)
Truck-tractor and semitrailer	156	106	65	28	13	2
Truck and trailer	316	226	117	98	40	25
Average, truck combinations	163	112	70	31	14	3
Average, all trucks and combinations	63	43	27	13	6	1
East South Central:						
2-axle, 4-tire	9	9	4	4	4	---
2-axle, 6-tire	13	7	4	2	1	---
3-axle	66	43	11	---	---	---
Average, single-unit trucks	7	4	2	1	1	---
Truck-tractor and semitrailer	162	118	81	26	9	1
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	162	118	81	26	9	1
Average, all trucks and combinations	36	25	17	6	2	(1)
West North Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	7	2	1	1	---	---
3-axle	56	37	24	8	---	---
Average, single-unit trucks	5	2	1	1	---	---
Truck-tractor and semitrailer	179	123	76	29	12	1
Truck and trailer	20	16	8	6	---	---
Average, truck combinations	173	119	74	28	12	1
Average, all trucks and combinations	46	30	19	8	3	(1)
West South Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	10	7	4	1	---	---
3-axle	---	---	---	---	---	---
Average, single-unit trucks	4	3	2	(1)	---	---
Truck-tractor and semitrailer	153	115	76	31	11	5
Truck and trailer	10	10	---	---	---	---
Average, truck combinations	149	112	74	30	11	5
Average, all trucks and combinations	42	31	21	8	3	1
Mountain:						
2-axle, 4-tire	2	2	2	2	2	---
2-axle, 6-tire	34	31	19	11	7	(-)
3-axle	63	55	48	38	34	---
Average, single-unit trucks	15	13	8	5	4	(1)
Truck-tractor and semitrailer	215	167	120	64	27	5
Truck and trailer	233	179	121	69	25	15
Average, truck combinations	218	169	120	65	27	7
Average, all trucks and combinations	58	46	32	18	9	1
Pacific:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	6	4	2	1	1	1
3-axle	18	7	1	(1)	---	---
Average, single-unit trucks	4	2	1	(1)	(1)	(1)
Truck-tractor and semitrailer	116	63	36	9	3	(1)
Truck and trailer	171	50	16	2	(1)	---
Average, truck combinations	133	59	30	7	2	(1)
Average, all trucks and combinations	49	22	11	2	1	(1)
United States average:						
2-axle, 4-tire	1	1	1	(1)	(1)	---
2-axle, 6-tire	13	9	6	2	1	(1)
3-axle	75	61	42	24	13	4
Average, single-unit trucks	8	6	4	2	1	(1)
Truck-tractor and semitrailer	158	109	71	29	13	3
Truck and trailer	193	107	70	33	12	8
Average, truck combinations	160	109	71	29	13	3
Average, all trucks and combinations	51	35	23	10	4	1
Average, 1948	55	38	26	12	6	1
Average, 1947	46	34	23	10	4	1

[1] Less than 5 per 10,000.

[2] Data omitted because of insufficient sample.

Table 11.—*Number of axles, per 1,000 loaded and empty trucks and truck combinations, that exceeded the permissible axle-load limit of 18,000 pounds recommended by the A.A.S.H.O. by various percentages of overload in the summer of 1949*

Region and type of vehicle (panel and pickup trucks excluded)	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	44	35	30	19	11	3
3-axle	91	46	33	---	---	---
Average, single-unit trucks	26	20	17	10	6	2
Truck-tractor and semitrailer	403	337	259	118	44	4
Truck and trailer	(1)	(1)	(1)	---	---	---
Average, truck combinations	408	343	261	117	44	4
Average, all trucks and combinations	109	90	70	33	14	2
Middle Atlantic:						
2-axle, 4-tire	1	1	---	---	---	---
2-axle, 6-tire	54	47	40	21	9	2
3-axle	175	129	70	35	21	7
Average, single-unit trucks	36	30	24	13	6	1
Truck-tractor and semitrailer	480	384	312	177	89	24
Truck and trailer	(1)	(1)	(1)	(1)	---	---
Average, truck combinations	481	385	313	177	89	24
Average, all trucks and combinations	175	141	115	64	32	8
South Atlantic:						
2-axle, 4-tire	1	1	1	---	---	---
2-axle, 6-tire	17	13	9	3	2	(2)
3-axle	55	41	27	8	---	---
Average, single-unit trucks	10	7	5	2	1	(2)
Truck-tractor and semitrailer	299	229	160	62	21	3
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	299	229	160	62	21	3
Average, all trucks and combinations	90	68	48	19	7	1
East North Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	7	5	3	1	---	---
3-axle	34	34	31	28	20	---
Average, single-unit trucks	5	3	2	1	1	---
Truck-tractor and semitrailer	165	104	60	22	8	1
Truck and trailer	406	247	151	63	17	---
Average, truck combinations	176	111	64	24	8	1
Average, all trucks and combinations	67	42	25	9	4	(2)
East South Central:						
2-axle, 4-tire	9	9	4	4	4	---
2-axle, 6-tire	13	7	4	2	1	---
3-axle	33	11	---	---	---	---
Average, single-unit trucks	7	4	2	1	1	---
Truck-tractor and semitrailer	191	131	79	22	7	1
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	191	131	79	22	7	1
Average, all trucks and combinations	41	28	16	5	2	(2)
West North Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	7	2	1	(2)	---	---
3-axle	21	10	2	2	---	---
Average, single-unit trucks	4	1	1	(2)	---	---
Truck-tractor and semitrailer	168	94	51	14	5	1
Truck and trailer	9	9	5	4	---	---
Average, truck combinations	162	91	49	14	5	1
Average, all trucks and combinations	42	23	13	3	1	(2)
West South Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	10	7	4	1	---	---
3-axle	---	---	---	---	---	---
Average, single-unit trucks	4	3	2	(2)	---	---
Truck-tractor and semitrailer	180	122	73	27	11	2
Truck and trailer	10	10	---	---	---	---
Average, truck combinations	175	119	71	26	11	2
Average, all trucks and combinations	49	33	20	7	3	1
Mountain:						
2-axle, 4-tire	2	2	2	2	2	---
2-axle, 6-tire	33	31	19	11	7	(2)
3-axle	67	67	67	27	13	---
Average, single-unit trucks	14	14	9	5	3	(2)
Truck-tractor and semitrailer	216	158	112	51	21	4
Truck and trailer	132	75	39	15	7	2
Average, truck combinations	201	143	99	44	18	4
Average, all trucks and combinations	54	41	28	13	6	1
Pacific:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	6	4	2	1	1	1
3-axle	2	---	---	---	---	---
Average, single-unit trucks	3	2	1	(2)	(2)	(2)
Truck-tractor and semitrailer	70	37	16	3	1	(2)
Truck and trailer	113	40	12	2	(2)	---
Average, truck combinations	83	38	14	3	1	(2)
Average, all trucks and combinations	31	15	6	1	(2)	(2)
United States average:						
2-axle, 4-tire	1	1	1	(2)	(2)	---
2-axle, 6-tire	19	15	11	5	3	(2)
3-axle	57	43	28	14	8	1
Average, single-unit trucks	11	8	6	3	2	(2)
Truck-tractor and semitrailer	242	175	122	55	24	5
Truck and trailer	168	92	50	17	4	(2)
Average, truck combinations	238	171	118	53	23	5
Average, all trucks and combinations	75	54	37	17	8	1
Average, 1948	85	63	45	23	11	2
Average, 1947	66	49	33	15	7	1

[1] Data omitted because of insufficient sample.

[2] Less than 5 per 10,000.

Table 12.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle-group loads recommended by the A.A.S.H.O. by various percentages of overload in the summer of 1949*

Region and type of vehicle (panel and pickup trucks excluded)	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	1	1	1	(1)	---	---
3-axle	106	81	48	24	6	---
Average, single-unit trucks	3	3	2	1	(1)	---
Truck-tractor and semitrailer	68	42	24	10	4	(1)
Truck and trailer	(2)	(2)	(2)	(2)	(2)	---
Average, truck combinations	71	45	27	14	8	(1)
Average, all trucks and combinations	18	12	7	4	2	(1)
Middle Atlantic:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	1	---	---	---	---	---
3-axle	157	129	112	45	14	7
Average, single-unit trucks	5	4	3	1	(1)	(1)
Truck-tractor and semitrailer	142	102	73	41	25	5
Truck and trailer	(2)	(2)	(2)	---	---	---
Average, truck combinations	144	104	75	41	25	5
Average, all trucks and combinations	49	35	26	14	8	2
South Atlantic:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	---	---	---	---	---	---
3-axle	62	34	21	14	8	---
Average, single-unit trucks	1	1	(1)	(1)	(1)	---
Truck-tractor and semitrailer	54	39	29	16	9	1
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	54	39	29	16	9	1
Average, all trucks and combinations	16	12	8	4	3	(1)
East North Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	---	---	---	---	---	---
3-axle	48	35	35	29	29	10
Average, single-unit trucks	1	1	1	1	1	(1)
Truck-tractor and semitrailer	98	75	50	25	11	3
Truck and trailer	425	407	374	293	191	56
Average, truck combinations	113	90	65	37	19	6
Average, all trucks and combinations	42	33	24	14	8	2
East South Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	---	---	---	---	---	---
3-axle	11	---	---	---	---	---
Average, single-unit trucks	(1)	---	---	---	---	---
Truck-tractor and semitrailer	25	15	9	2	1	(1)
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	25	15	9	2	1	(1)
Average, all trucks and combinations	5	3	2	(1)	(1)	(1)
West North Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	---	---	---	---	---	---
3-axle	15	8	8	---	---	---
Average, single-unit trucks	(1)	(1)	(1)	---	---	---
Truck-tractor and semitrailer	83	58	36	11	4	(1)
Truck and trailer	16	6	6	2	---	---
Average, truck combinations	81	56	35	11	4	(1)
Average, all trucks and combinations	20	14	9	3	1	(1)
West South Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	---	---	---	---	---	---
3-axle	---	---	---	---	---	---
Average, single-unit trucks	---	---	---	---	---	---
Truck-tractor and semitrailer	52	36	22	11	6	2
Truck and trailer	10	10	10	10	10	---
Average, truck combinations	51	35	22	11	6	2
Average, all trucks and combinations	13	9	6	3	2	1
Mountain:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	(1)	(1)	(1)	---	---	---
3-axle	53	50	38	33	15	---
Average, single-unit trucks	1	1	1	1	(1)	---
Truck-tractor and semitrailer	160	122	92	43	19	3
Truck and trailer	246	185	126	47	29	12
Average, truck combinations	176	134	98	44	21	5
Average, all trucks and combinations	38	29	22	10	4	1
Pacific:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	---	---	---	---	---	---
3-axle	11	5	---	---	---	---
Average, single-unit trucks	1	(1)	---	---	---	---
Truck-tractor and semitrailer	143	111	71	22	5	(1)
Truck and trailer	258	178	74	5	(1)	---
Average, truck combinations	179	132	72	17	3	(1)
Average, all trucks and combinations	64	46	25	6	1	(1)
United States average:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	(1)	(1)	(1)	(1)	---	---
3-axle	58	43	34	19	10	3
Average, single-unit trucks	1	1	1	(1)	(1)	(1)
Truck-tractor and semitrailer	89	65	44	21	10	2
Truck and trailer	260	207	140	72	46	13
Average, truck combinations	98	73	49	24	12	3
Average, all trucks and combinations	28	21	14	7	3	1
Average, 1948	30	22	16	7	3	1
Average, 1947	25	19	12	5	2	1

[1] Less than 5 per 10,000.

[2] Data omitted because of insufficient sample.

Table 13.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded any of the permissible load limits recommended by the A.A.S.H.O. by various percentages (maximum) of overload in the summer of 1949*

Region and type of vehicle (panel and pickup trucks excluded)	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	44	36	30	19	11	3
3-axle	119	86	57	24	6	---
Average, single-unit trucks	26	21	17	11	6	2
Truck-tractor and semitrailer	288	250	203	108	45	4
Truck and trailer	(1)	(1)	(1)	(1)	(1)	---
Average, truck combinations	290	252	206	111	48	4
Average, all trucks and combinations	84	71	58	33	15	2
Middle Atlantic:						
2-axle, 4-tire	1	1	---	---	---	---
2-axle, 6-tire	54	47	40	21	9	2
3-axle	181	149	112	53	21	7
Average, single-unit trucks	36	31	26	13	6	1
Truck-tractor and semitrailer	345	284	244	153	85	26
Truck and trailer	(1)	(1)	(1)	(1)	---	---
Average, truck combinations	346	285	245	153	85	26
Average, all trucks and combinations	133	111	95	57	31	9
South Atlantic:						
2-axle, 4-tire	1	1	1	---	---	---
2-axle, 6-tire	17	13	9	3	2	(2)
3-axle	66	37	24	14	8	---
Average, single-unit trucks	10	7	5	2	1	(2)
Truck-tractor and semitrailer	229	182	138	64	26	4
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	229	182	138	64	26	4
Average, all trucks and combinations	71	55	42	19	8	1
East North Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	7	5	3	1	---	---
3-axle	56	43	43	38	29	10
Average, single-unit trucks	5	4	3	1	1	(2)
Truck-tractor and semitrailer	180	133	86	39	18	3
Truck and trailer	425	414	381	293	191	56
Average, truck combinations	191	146	100	51	26	6
Average, all trucks and combinations	73	56	38	19	10	2
East South Central:						
2-axle, 4-tire	9	9	4	4	4	---
2-axle, 6-tire	13	7	4	2	1	---
3-axle	33	11	---	---	---	---
Average, single-unit trucks	7	4	2	1	1	---
Truck-tractor and semitrailer	151	111	74	22	8	1
Truck and trailer	---	---	---	---	---	---
Average, truck combinations	151	111	74	22	8	1
Average, all trucks and combinations	35	24	15	5	2	(2)
West North Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	7	2	1	1	---	---
3-axle	36	17	10	2	---	---
Average, single-unit trucks	4	1	1	1	---	---
Truck-tractor and semitrailer	172	112	65	20	8	1
Truck and trailer	16	8	8	6	---	---
Average, truck combinations	167	108	63	20	8	1
Average, all trucks and combinations	44	27	16	6	2	(2)
West South Central:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	10	7	4	1	---	---
3-axle	---	---	---	---	---	---
Average, single-unit trucks	4	3	2	(2)	---	---
Truck-tractor and semitrailer	149	113	73	30	12	3
Truck and trailer	10	10	10	10	10	---
Average, truck combinations	145	110	72	29	12	3
Average, all trucks and combinations	41	31	20	8	3	1
Mountain:						
2-axle, 4-tire	2	2	2	2	2	---
2-axle, 6-tire	34	31	19	11	7	(2)
3-axle	63	60	48	34	24	---
Average, single-unit trucks	15	13	8	5	3	(2)
Truck-tractor and semitrailer	233	185	139	74	35	6
Truck and trailer	258	198	136	55	29	15
Average, truck combinations	238	187	138	71	34	8
Average, all trucks and combinations	62	50	36	19	10	2
Pacific:						
2-axle, 4-tire	---	---	---	---	---	---
2-axle, 6-tire	6	4	2	1	1	1
3-axle	13	5	(2)	---	---	---
Average, single-unit trucks	4	2	1	(2)	(2)	(2)
Truck-tractor and semitrailer	166	127	80	22	6	1
Truck and trailer	299	212	83	7	(2)	---
Average, truck combinations	207	153	81	17	4	1
Average, all trucks and combinations	75	55	29	6	1	(2)
United States average:						
2-axle, 4-tire	1	1	1	(2)	(2)	---
2-axle, 6-tire	19	15	11	6	3	(2)
3-axle	68	50	38	22	12	3
Average, single-unit truck	11	9	6	3	2	(2)
Truck-tractor and semitrailer	210	162	119	58	28	6
Truck and trailer	281	227	148	75	46	14
Average, truck combinations	214	165	121	59	29	6
Average, all trucks and combinations	68	53	38	19	10	2
Average, 1948	73	56	42	23	11	3
Average, 1947	59	45	32	15	7	1

[1] Data omitted because of insufficient sample.

[2] Less than 5 per 10,000.

Table 14.—*Number of trucks and truck combinations per 1,000 loaded and empty vehicles, in private and in for-hire operation, that exceeded various load limits by various percentages of overload in the summer of 1949 (U. S. average)*

Type of vehicle	Private operation						For-hire operation					
	Number per 1,000 over-loaded	Number per 1,000 overloaded more than—					Number per 1,000 over-loaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent		5 percent	10 percent	20 percent	30 percent	50 percent
NUMBER OF TRUCKS AND TRUCK COMBINATIONS PER 1,000 EXCEEDING PERMISSIBLE AXLE, AXLE-GROUP, OR GROSS-WEIGHT LEGAL LIMITS OF THE SEVERAL STATES												
2-axle, 4-tire	1	1	1	1	(1)	------	-------	-------	-------	------	------	------
2-axle, 6-tire	11	7	5	2	1	------	23	16	10	6	3	(1)
3-axle	65	53	39	24	14	5	99	82	51	27	10	------
Average, single-unit trucks	6	4	3	1	1	(1)	27	20	12	7	3	(1)
Truck-tractor and semitrailer	148	103	70	29	12	3	164	112	71	29	14	3
Truck and trailer	111	61	42	25	20	15	247	140	88	38	6	3
Average, truck combinations	146	101	69	29	12	4	168	114	72	29	14	3
Average, all trucks and combinations	26	18	13	5	3	1	131	89	56	23	11	2
NUMBER OF AXLES PER 1,000 TRUCKS AND TRUCK COMBINATIONS EXCEEDING THE 18,000-POUND LIMIT RECOMMENDED BY THE A. A. S. H. O.												
2-axle, 4-tire	1	1	1	(1)	(1)	------	12	12	-------	------	------	------
2-axle, 6-tire	17	12	9	4	2	(1)	36	29	23	14	6	2
3-axle	40	29	20	9	5	------	97	76	47	27	15	4
Average, single-unit trucks	9	6	5	2	1	(1)	38	31	23	14	6	2
Truck-tractor and semitrailer	222	159	110	45	18	4	255	186	130	61	28	6
Truck and trailer	136	82	50	20	6	1	190	97	50	14	3	------
Average, truck combinations	217	155	107	44	17	4	251	181	126	58	27	6
Average, all trucks and combinations	39	28	20	8	3	1	195	141	99	46	21	5
NUMBER OF TRUCKS AND TRUCK COMBINATIONS PER 1,000 EXCEEDING THE MAXIMUM AXLE-GROUP LOADS RECOMMENDED BY THE A. A. S. H. O.												
2-axle, 6-tire	(1)	(1)	(1)	(1)	------	------	1	(1)	(1)	------	------	------
3-axle	43	28	24	14	9	3	92	77	60	28	12	4
Average, single-unit trucks	1	(1)	(1)	(1)	(1)	(1)	8	6	5	2	1	(1)
Truck-tractor and semitrailer	71	54	36	18	9	2	102	74	50	23	11	3
Truck and trailer	135	113	89	54	45	20	341	270	175	87	50	11
Average, truck combinations	74	57	39	20	11	3	115	85	57	26	13	3
Average, all trucks and combinations	12	8	6	3	2	(1)	87	64	43	20	10	2
NUMBER OF TRUCKS AND TRUCK COMBINATIONS PER 1,000 EXCEEDING ANY OF THE MAXIMUM MOTOR-VEHICLE LOADS RECOMMENDED BY THE A. A. S. H. O.												
2-axle, 4-tire	1	1	1	(1)	(1)	------	12	12	-------	------	------	------
2-axle, 6-tire	16	12	9	4	2	(1)	36	29	23	16	6	2
3-axle	53	36	25	14	9	3	99	80	63	36	18	4
Average, single-unit trucks	8	6	5	2	1	(1)	38	31	24	16	6	2
Truck-tractor and semitrailer	185	142	104	50	21	5	228	177	129	64	31	7
Truck and trailer	140	121	95	51	41	17	372	296	182	90	49	11
Average, truck combinations	183	141	104	50	22	6	236	183	132	65	32	7
Average, all trucks and combinations	34	26	19	9	4	1	184	143	103	52	25	6

[1] Less than 5 per 10,000.

Report of Trends in Motor-Vehicle Travel Discontinued

For several years past, PUBLIC ROADS has annually carried a companion article, *Trends in motor-vehicle travel,* which provided estimates of total urban and rural travel and average unit travel and fuel consumption of each of the major classes of motor vehicles.

In making these estimates it has been necessary to predicate the estimates of a given year on the sequence of values obtained for previous years, with the 1936–37 period of the comprehensive State-wide traffic surveys as a base. As this base period has become more and more remote the hazards of the procedure have multiplied.[1]

The interval of nearly 15 years and the changes in circumstances affecting travel are so great that the estimates can no longer be published with confidence. It is hoped that they can be resumed when the States have accumulated a sufficient body of current basic data.

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[1] A fuller explanation will be found in *Trends in motor-vehicle travel, 1947,* PUBLIC ROADS, vol. 25, No. 3, Mar. 1949, p. 160.

(*Continued from page 104*)

described here is applicable to such hydraulic materials as pozzolanic, slag, and natural cements which cannot be analyzed accurately by the tentative method for portland cements.

A further application of this method would be for limestone or other calcareous material even though certain types may be completely soluble in mineral acid. By using the J. Lawrence Smith fusion on such materials, the eventual calcium in solution would be the same as that obtained for the siliceous samples. Thus, no previous knowledge of the calcium content of the sample would be required and the same correction curves would be applicable.

Flame-Photometer Determination of Sodium and Potassium in Soils and Other Siliceous Materials

BY THE PHYSICAL
RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by W. J. Halstead, Chemist, and Bernard Chaiken, Associate Chemist

This article presents a method for using a flame photometer to determine the alkalies—sodium and potassium oxides—in soil and other siliceous material which cannot easily be decomposed by mineral acids. The method is much more rapid than existing gravimetric procedures, and yields results which are equally accurate.

The sample is decomposed by fusion and the melt is leached with hot water in order to bring the alkalies into solution as chlorides. The concentration of alkalies in the resulting solution is then determined by means of a flame photometer, using the direct-intensity method.

A wide variety of materials can be analyzed by the method, including soils, sands, rocks, various minerals, ceramic clays, fly ash, and pozzolanic, slag, and natural cements. The method is also applicable to the determination of the alkalies in limestones and other calcareous materials.

GRAVIMETRIC METHODS for determining the alkalies—sodium, lithium, and potassium—are long and tedious. The lengthy operations required may introduce serious errors unless meticulous care is observed throughout the entire procedure. For this reason, there has long been much effort directed toward developing an alternate procedure for the determination of these elements. This has been accomplished by the recent developments in flame photometry. Successful and rapid flame photometric methods for the determination of sodium and potassium in such complex materials as whole blood, serum, urine, fertilizers, portland cements (*1, 4, 7*),[1] plant tissues (*10, 11*), soil extracts, and cation-exchange capacity of soils (*8, 9*), have been described elsewhere.

The fundamental basis of flame photometry is in the long-known fact that all elements, when heated to a sufficiently high temperature, will emit characteristic light waves of definite wave lengths. The intensity of these emissions varies directly with the quantity of the element present. The temperature necessary to cause the emissions for the various elements ranges from the extremely high value which may be obtained with an electric arc to the comparatively low temperature of a gas flame. The flame photometer is applicable to those elements which emit radiations at the temperature of an acetylene or propane flame. Essentially, it is an instrument by which the desired element is introduced into a gas flame under carefully controlled conditions, and the characteristic wave length of the element is isolated and its intensity is measured photoelectrically.

The alkali metals—lithium, sodium, and potassium—emit characteristic radiations at the temperature of the propane flame and this gas is generally used for the determination of these elements. The flame photometer is also applicable to the determination of calcium, strontium, barium, magnesium, manganese, and chromium, provided a higher temperature flame, such as that of acetylene, is used.

Summary

This article describes a method for using the flame photometer to determine sodium and potassium in various siliceous materials which cannot easily be decomposed by acids. These include such materials as soils, sands, rocks, various minerals, ceramic clays, fly ash, pozzolanic cement, natural cement, and slag cement. The alkalies are brought into solution by the J. Lawrence Smith fusion method with calcium carbonate, and the melt leached with hot water as is customary. The concentrations of the sodium and potassium oxides in the resulting solution are then determined with a flame photometer, using the direct-intensity procedure. Tests show that calcium is the only interfering ion present. By careful quantitative control of the reagents and control of the amount of washings, concentration of the calcium ion can be held substantially constant at 1,700 parts per million as calcium oxide. Correction can then be made for the effect of this constituent by the use of correction curves. This makes possible the calibration against standard solutions containing only sodium and potassium chlorides, which is advantageous since such solutions are used in a large number of other analyses.

In addition to soils, sands, various minerals, ceramic clays, fly ash, and similar siliceous materials, the flame-photometer method is applicable to the determination of the alkalies in the nonportland type cements such as pozzolanic, slag, and natural cements. While actual tests were not made, the method is indicated as also applicable to limestone and other calcareous materials. The advantage of the method for this type of material is that a neutral solution of a definite concentration of calcium oxide is obtained regardless of the calcium content of the limestone or calcareous material being analyzed.

Comparisons of the results obtained with the flame-photometer method and the results of gravimetric methods show the average differences in percentage to be less than 0.1 when 0.5-gram samples are used. This is of the same order as differences between two gravimetric results on the same sample. The time saved by the flame-photometer method is considerable. Comparative records show that for the same number of analyses only one-fourth the time is required for the flame-photometer method (including the time for calibration) as is required by the usual gravimetric procedure.

Many features of the given flame-photometer method follow well-established procedures. However, the quantitative evaluation of the interferences from other elements and the technique of the detailed operations as given in this article should be of considerable value to those engaged in analyzing such materials or to those who may be seeking further applications of the flame photometer.

Direct-Intensity Procedure

The instrument used in this study was a model 52A Perkin–Elmer flame photometer, which has been described in a number of publications (*1, 7*). This flame photometer can be used in either of two ways, both of which require careful calibration against standard solutions having a range of concentrations of the desired element overlapping that to be expected in the unknown samples.

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[1] Italic numbers in parentheses refer to the bibliographic reference list on page 104.

These two procedures are known as the internal-standard method and the direct-intensity method.

In the internal-standard method, the emitted light-intensity ratio of the element sought and another element added in known amount (the internal standard) is measured. This is done by amplifying the photoelectric current resulting from the unknown until it equals that from the internal standard, the amount of amplification required being indicated by the reading of the gain-control dial. A basic requirement of the internal-standard method is that the element chosen as the internal standard be absent from the sample to be analyzed. The flame photometer used in this work is so equipped as to require the use of a lithium salt exclusively as the internal standard. However, this element is often present in materials of mineral origin (*5*, page 517) or highly sodic rocks (*12*). Thus, in analyzing a wide variety of siliceous materials, the use of the internal-standard method is not desirable.

In the direct-intensity method, a measure of the absolute light-intensity of the element sought is obtained directly by the meter reading and there is no interference from lithium when present.

Method of Analysis

The method described in this article is directly concerned with those materials which are not easily or completely decomposed by mineral acids, and thus require fusion or special digestion in order to bring the alkalies completely into solution.

The familiar J. L. Smith method of fusion with calcium carbonate, as described by Hillebrand and Lundell (*5*, page 787), was chosen, since its use limits the metals which go into solution. After the fusion the melt is extracted with water in the usual manner and the resulting solution is used for the determinations of the alkalies by means of the flame photometer. The adjustment and calibration of the apparatus follow closely the directions given in the tentative method of test for sodium oxide and potassium oxide in portland cement by flame photometry of the American Society for Testing Materials (*1*), the chief differences being that the standard solutions used in this method contain only sodium and potassium chloride, and the zero adjustment is made with distilled water.

By careful control of the weights of the sample and the reagents, and of the volume of wash water used, it was found that the amount of calcium in solution could be held substantially constant, and thus the results could be corrected for any interference resulting from this element.

In order to present a complete picture, the method of test, including the calibration of the instrument, the determination of the correction curve, and the preparation of the sample, is given here in detailed step-by-step procedure.

1. Reagents

(*a*) Calcium carbonate: A.C.S. "low-alkali" $CaCO_3$, limited to 0.02 percent total alkalies as sulfate.

(*b*) Sodium chloride: A.C.S. NaCl with a maximum limit of 0.01 percent potassium.

(*c*) Potassium chloride: A.C.S. KCl with a maximum limit of 0.02 percent sodium.

(*d*) Ammonium chloride: Reagent grade A.C.S. NH_4Cl.

(*e*) Brom-thymol blue indicator: 0.04 percent solution in water.

(*f*) Water: Distilled water.

2. Solutions

(*a*) *Sodium-potassium chloride stock solution.*—Prepare a solution containing 1.8858 gm. of NaCl and 1.5830 gm. of KCl (previously dried at 105° C. for several hours) dissolved in water and diluted to 1 liter in a volumetric flask. This solution contains the equivalent of 1,000 p.p.m. (parts per million) each of Na_2O and K_2O.

(*b*) *Calcium chloride stock solution.*—Prepare a solution containing 6.068 gm. of $CaCO_3$ per liter as follows: Weigh the $CaCO_3$ into a large beaker. Add sufficient water to form a slurry and then add concentrated HCl cautiously until the $CaCO_3$ is dissolved. Avoid any excess of HCl by adding the acid drop by drop with vigorous stirring until the solution just clears. Cool the solution to room temperature and filter into a 1,000-ml. volumetric flask. Dilute to the mark and mix thoroughly. This solution contains the equivalent of 3,400 p.p.m. of CaO.

(*c*) *Standard sodium-potassium chloride solutions.*—Using the NaCl-KCl stock solution, prepare standard solutions containing the equivalent of 5, 20, 40, 60, 80, and 100 p.p.m. each of Na_2O and K_2O. Store these solutions in acid-resistant glass bottles with ground-glass or rubber stoppers.

(*d*) *Correction solutions.*—Using the $CaCl_2$ stock solution and the NaCl-KCl stock solution, prepare solutions containing the equivalent of 0, 5, 20, 40, 60, 80, and 100 p.p.m. of Na_2O and K_2O, respectively, and each containing the equivalent of 1,700 p.p.m. of CaO.

3. Calibration of flame photometer

(*a*) Turn on the instrument, adjust the air pressure to 10 p.s.i. and the propane gas pressure to 5 p.s.i. Adjust the burner so as to give a faintly visible flame 5 to 6 inches high and with ⅛-inch cones over the burner grid. These cones should be uniform, quiet, and of a blue or greenish-blue color. Allow the system to warm up for approximately 30 minutes after the current and gas are turned on.

(*b*) Set the internal-standard dial at zero and adjust the meter reading to zero with the zero adjustment knob.

(*c*) Find the correct position on the wave-length dial for the element to be determined by pouring into the atomizer a portion of the 100-p.p.m. standard solution of Na_2O-K_2O and moving the selector slowly back and forth on each side of the indicated wave length for the element until the point of maximum deflection is noted. Set the wave-length selector at this point. The coarse and fine gain controls are used to adjust the deflections to the range of 90 to 100.

(*d*) Pour the 100-p.p.m. standard solution into the atomizer and adjust the controls until the meter reading is 100. Then pour in distilled water and adjust the zero-adjustment knob until the meter reads zero. Repeat these two operations until no adjustment is necessary in going from one to the other.

(*e*) Next pour into the atomizer the 80-p.p.m. standard solution and note the meter reading to the nearest whole division.[2] Check the zero reading with distilled water and the 100 reading with the 100-p.p.m. standard solution. If these readings are exactly zero and within one scale division of 100, respectively, the reading for the 80-p.p.m. solution can be considered correct. If the zero reading is not exact, or the 100-p.p.m. reading is not within one scale division of 100, repeat the adjustments in step 3 (*d*) and take another reading for the 80-p.p.m. solution. Continue until no adjustment of the zero and 100 points is necessary after securing the reading for the 80-p.p.m. standard solution.

(*f*) In a similar manner, determine the readings[2] for the 60, 40, 20, and 5-p.p.m. standard solutions of Na_2O-K_2O.

(*g*) Plot calibration curves for each oxide, using cross-section paper of such type that each division on the ordinate represents a meter reading of one unit and each division on the abscissa represents a concentration of 1 p.p.m. (see fig. 1, page 101).

4. Correction curves

(*a*) Using the correction solutions, determine the apparent Na_2O and K_2O concentration in parts per million as directed for the analysis of the samples in steps 6 and 7, with the application of the calibration curve as determined in accordance with step 3 (*g*).

(*b*) To determine the reagent impurities, make a blank determination, following the procedure outlined in step 5 for the preparation of the sample and in steps 6 and 7 for the determination of Na_2O and K_2O. The meter readings obtained are converted to parts per million (see step 3 *g*) and the latter values represent the effect of the 1,700-p.p.m. CaO in addition to the reagent impurities. Subtract from the value for each oxide the value obtained for the correction solution containing no added NaCl or KCl. These values represent the effect of the reagent impurities alone.

(*c*) Add the effect of the reagent impurities as obtained in step 4 (*b*) to the apparent sodium and potassium concentration obtained in step 4 (*a*). Plot the resulting apparent concentrations as abscissas and the known true value for the alkali oxide as ordinates. This gives a correction curve which includes the reagent blank. Once obtained, it is not necessary to recheck the curve except when the reagents are changed or when some change is made in the instrument.

5. Preparation of sample

(*a*) Place 0.5000 ± 0.0005 gm. of finely ground sample (passing No. 100 sieve) into an agate mortar, add 0.500 gm. NH_4Cl, and grind until completely mixed.[3] Add 4.0 gm. $CaCO_3$ to the mixture, grind until well mixed, and transfer the mixture to a J. Lawrence

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[2] For meter readings of all solutions in the lower half of the scale, the apparatus is sufficiently stable and sensitive so that readings may be made to the nearest half division or less.

[3] Close control of the quantity of NH_4Cl is necessary since it directly affects the amount of calcium which goes into solution as the chloride.

Smith platinum crucible containing approximately 0.5 gm. $CaCO_3$ in the bottom. Rinse the mortar and pestle with approximately 0.5 gm. $CaCO_3$ and add to the crucible. Cap the crucible, tap it gently to cause the powder to settle, and place it in a slightly inclined position in a refractory cylinder provided with a suitable hole to receive the crucible.

(*b*) Heat the portion of the crucible within the cylinder by means of a fish-tail flame placed well below the crucible for about 15 minutes or until the odor of ammonia is no longer perceptible. The heat should not be strong enough to cause vapors of NH_4Cl to escape. The crucible should be rotated at intervals during the early stages of the ignition. Gradually increase the heat until the crucible is a bright red and maintain this temperature for 40 to 60 minutes.[4] Allow the crucible to cool and transfer the sintered mass to a 250-ml. casserole. Pour hot water into the crucible and digest until all remaining matter can be washed out into the casserole or until it is thoroughly extracted. Slake the sintered cake in the casserole by adding cautiously a few milliliters of water, then add approximately 50 ml. more and digest on the steam bath until the cake is thoroughly disintegrated (4 to 8 hours). Disintegration may be aided by grinding with an agate pestle during the digestion period.

(*c*) After the cake has completely disintegrated, adjust the volume of liquid in the casserole to approximately 50 ml., evaporating if necessary. Heat the covered casserole to boiling on a hot plate, let the solids settle, and rapidly filter into a 250-ml. volumetric flask made of acid-resistant glass. Add 30 ml. of hot water to the casserole, break up any lumps by gentle pressure with a pestle or glass stirring rod, heat to boiling, let settle, and filter. Repeat the extraction with 30 ml. of water three more times (a total of four 30-ml. washings). Transfer the bulk of the residue to the filter paper with a minimum amount of hot water, and wash the residue on the paper once or twice with hot water so as to bring the filtrate level to within several centimeters of the neck of the flask.[5]

Add three or four drops of brom-thymol blue indicator to the flask and then add concentrated HCl drop by drop until one drop causes the indicator to change color. Cool the flask and its contents to room temperature, add sufficient water to bring the liquid level to the calibration mark, and mix well. This solution is then used for the flame-photometer analysis.

6. Determination of sodium oxide

(*a*) Warm up the apparatus and calibrate for the sodium determination, following the instructions given in step 3.

(*b*) Add a portion of the sample solution to the atomizer and record the meter reading to the nearest whole division.[6] Then select the standard solution which gives a meter reading closest to the unknown and record its reading. This latter value should agree to within one division on the meter scale with the average value established during the calibration of the apparatus. If it does not, reset the meter needle to the original calibration point by use of the fine-gain control. Check the zero and 100 division points with the appropriate standard solutions. Finally, alternate the use of the unknown solution and the closest standard until readings for the unknown agree to within one division on the meter scale and the reading for the standard similarly agrees with the calibration point. Record the average of the last two meter readings obtained with the unknown solution.

7. Determination of Potassium Oxide

Follow the same procedure as for Na_2O (step 6) except that the instrument is calibrated with the wave-length selector set at the point of maximum response to K_2O by use of the 100-p.p.m. standard solution.

8. Calculation of results [7]

(*a*) From the recorded averages of the meter readings for Na_2O and K_2O read the concentration to the nearest half division from the calibration curve (step 3*g*). This is the apparent concentration of the alkali oxide in parts per million. Convert this value to the corrected or true concentration by use of the correction curve obtained in step 4.

(*b*) Calculate the percentage of alkali oxide to the nearest 0.02 percent as follows:

$$\text{Alkali oxide, percent} = \frac{C}{20}$$

Where C=true concentration in p.p.m. (from correction curve), and

$$20 = \frac{0.5 \text{ (weight of sample)}}{250 \text{ (volume of solution)}} \times \frac{1{,}000{,}000}{100}$$

Constant Calibration Check Needed

Careful calibration and a constant check on the calibration of the flame photometer are essential for accurate results. The calibration depends on the constancy of a large number of the system components, such as voltage, tube sensitivity, gain control, gas pressure, air pressure, atomizer efficiency, and burner adjustment. The calibration must be made before each run and certain calibration points checked during a run as directed. However, with the instrument operating normally when the zero and 100-p.p.m. points are properly set, the balance of the calibration points

Table 1.—*Example of data obtained in calibrating the flame photometer with standard solution* [1]

Concentration of Na_2O or K_2O in standard solution	Meter reading (in divisions) for—							
	Na_2O determination				K_2O determination			
	Test 1	Test 2	Test 3	Avg.	Test 1	Test 2	Test 3	Avg.
P.p.m.								
5	5	5	4.5	5	3	3	3	3
20	22	21	21	21	18	17	17	17
40	46.5	46.5	45.5	46	39.5	38.5	39.5	39
60	70.5	70	70	70	64	63.5	64	64
80	87	87	87.5	87	84	83	83	83
100	[2] 100	------	------	[2] 100	[2] 100	------	------	[2] 100

[1] The standard solutions were prepared as given in step 2(*c*) of the method of analysis, page 100.

[2] Apparatus was adjusted to give a full-scale deflection of 100 divisions with the standard solution containing 100 p.p.m. of the alkali oxide.

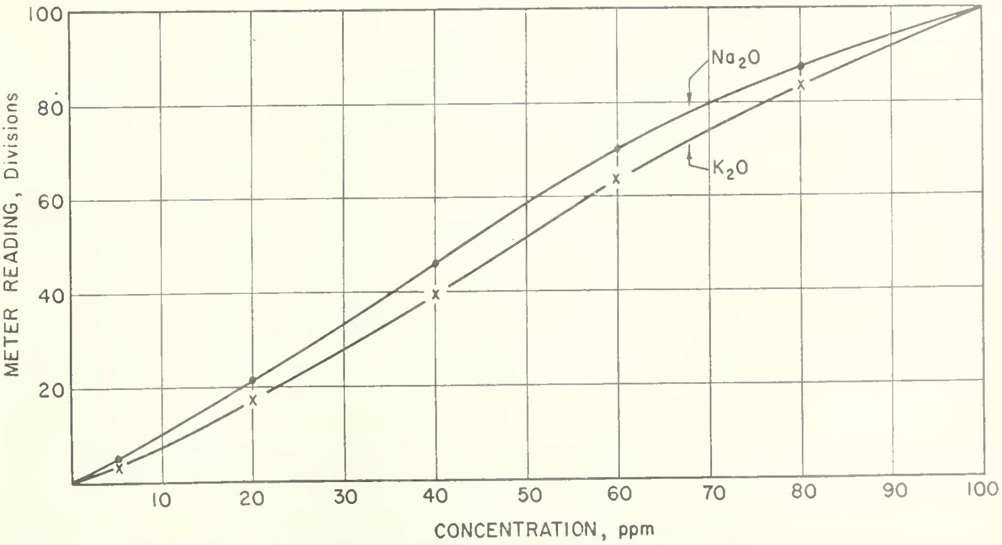

Figure 1.—Typical calibration curves for determination of sodium oxide and potassium oxide with the flame photometer, using the direct-intensity method.

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[4] In order to obtain a bright red heat, it may be necessary to replace the fish-tail burner with one of the Meker type.

[5] The final volume of the filtrate must be as nearly constant as possible in order to insure a uniform calcium concentration.

[6] See footnote 2, p. 100.

[7] These calculations are based on a 0.500-gm. sample. For samples containing greater than 5 percent of either alkali oxide a smaller sample must be used and the calculations varied accordingly.

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Table 2.—*Suggested form for recording test data in flame-photometer analyses*

Sample identification	Na_2O determination (a similar table is used for K_2O determination)						
	Meter reading (in divisions) of—				Concentration		
	Nearest standard	Unknown			Apparent	True	On basis of sample
		Test 1	Test 2	Average			
					P.p.m.	*P.p.m.*	*Percent*

usually remain essentially constant so that it may not be necessary to draw a new calibration curve for each run. Also, during the course of a run there is usually very little change in the calibration unless the atomizer is clogged or the burner flooded, in which cases the trouble is immediately apparent. Thus, the practical accomplishment of the conditions necessary for accurate results is not difficult.

Table 1 shows an example of the data obtained in calibrating the instrument. These data are plotted in figure 1. Table 2 presents a convenient form for recording the test data obtained in analyzing unknown samples.

Calcium Interference

The effects of the interference of other constituents on the alkali determination by flame photometry have been described (*3*, *6*) for instruments using color filters to isolate the wave lengths. However, the instrument used in this study employs a two-prism monochrometer for wave-length separation and, therefore, it is likely that the interferences may be different than when color filters are used.

The elements to be concerned with are sodium, potassium, and calcium. The other elements which may occur in natural silicates—iron, aluminum, barium, lithium, magnesium, manganese, strontium, and chromium—have a negligible effect. Furthermore, because of the method of decomposition employed, these constituents will not be present in solution except in very small amounts. Mineral and organic acids will not be present as such to interfere with the results.

Table 3 shows the effects of varying amounts of sodium, potassium, and calcium oxides on the determinations of Na_2O and K_2O. For these tests the instrument was calibrated and the apparent concentration of the alkali in the solution was determined as described in the method of analysis. The results indicate that for the type of solutions employed sodium and potassium do not interfere with each other within the range of concentration studied. However, the calcium ion (shown as CaO), when present in considerable amounts, has a significant inhibiting effect on determinations of both sodium and potassium.

A study was made to determine the concentration of calcium to be expected in test solutions. Aliquot portions were taken from a group of solutions prepared for the determination of the alkalies by the flame photometer. The amount of calcium in each solution was determined by a standard volumetric method. The results of these tests are given in table 4, the calcium being expressed as the equivalent amount of CaO. The range in these results was from approximately 1,600 to 1,800 p.p.m. with the average being 1,700. Thus, 1,700 p.p.m. of CaO was taken as being a suitable expression of the expected calcium concentration in all test solutions. Since the data shown in table 3 indicate that the effect of calcium does not change greatly with concentration, deviations from this average value which normally may be obtained will not be sufficient to make a measurable change in the interference of the calcium ion.

Table 3.—*Effect of certain constituents on the sodium oxide and potassium oxide determinations with the flame photometer*

[parts per million]

Interfering constituent and its concentration	Amount added: None		Amount added: 5 p.p.m.		Amount added: 80 p.p.m.	
	Amount found	Error	Amount found	Error	Amount found	Error
SODIUM OXIDE [1] DETERMINATION						
K_2O: [1]						
5	0	0	5	0	80	0
80	0	0	5	0	80	0
CaO: [2]						
50	0	0	5	0	80	0
300	0	0	5	0	79	−1
700	2	+2	5	0	75	−5
1,200	1	+1	6	+1	75	−5
1,700	2	+2	7	+2	75	−5
2,200	2	+2	7	+2	76	−4
POTASSIUM OXIDE [1] DETERMINATION						
Na_2O: [1]						
5	0	0	5	0	80	0
80	0	0	5	0	80	0
CaO: [2]						
50	0	0	5	0	77	−3
300	0	0	5	0	75	−5
700	0	0	5	0	74	−6
1,200	0	0	4	−1	73	−7
1,700	0	0	5	0	72	−8
2,200	0	0	4	−1	71	−9

[1] The proper amount of A.C.S. NaCl or KCl was dissolved in water to give the indicated oxide concentration.
[2] Present in solution as $CaCl_2$. The proper amount of A.C.S. "low-alkali" $CaCO_3$ was neutralized with just sufficient concentrated HCl to give the indicated oxide concentration.

Table 4.—*Concentration of calcium as CaO in test solutions of silicates prepared for flame-photometer analysis*

Sample identification	Type of material	Concentration of CaO in test solution [1]
		P. p. m.
19	Trap rock	1,606
20	Diorite	1,628
21	Fly ash	1,814
22	do	1,714
Blank *A*		1,708
Blank *B*		1,732
Average		1,700

[1] Volumetric determination with standard $KMnO_4$ made on aliquot portion.

A more complete study was made to show the effect of 1,700 p.p.m. of CaO on the sodium and potassium determination for the range of concentrations usually encountered. These analyses were made as before, using the calibration curve obtained with pure solutions of NaCl and KCl. The results are shown in table 5. These data further confirmed the conclusion drawn from the data in table 3 that, for this type of solution and within the range studied, the alkalies did not affect each other. Also, the interference of the calcium ion for a definite concentration of one alkali metal is the same regardless of the concentration of the other alkali. The inhibiting effect of the calcium ion increases as the concentration of the alkali increases. It should be noted that for the lower concentrations of Na_2O, 1,700 p.p.m. of CaO produces a positive error (see tables 3 and 5).

Correction Curve for Calcium

The use of a correction curve to correct for the effect of the calcium ion is a departure from the usual practice in flame-photometry applications. In the usual procedure, corrections would be made by using as standards the correction solutions, each of which contains 1,700 p.p.m. of CaO with different amounts of Na_2O and K_2O. A calibration curve constructed from data obtained with these solutions would then automatically correct for the interference. For laboratories concerned with the analysis of this type of silicate materials

Table 5.—*Effect of alkali oxides and 1,700 p.p.m. calcium oxide on the alkali determinations with the flame photometer*

[parts per million]

Na_2O determination								K_2O determination							
Amount of Na_2O in sample[1]	Apparent Na_2O determined when amount of K_2O added[1] (and 1,700 p.p.m. of CaO also added[2] in each case) was—						Average apparent Na_2O determined	Amount of K_2O in sample[1]	Apparent K_2O determined when amount of Na_2O added[1] (and 1,700 p.p.m. of CaO also added[2] in each case) was—						Average apparent K_2O determined
	0 p.p.m.	5 p.p.m.	20 p.p.m.	60 p.p.m.	80 p.p.m.	100 p.p.m.			0 p.p.m.	5 p.p.m.	20 p.p.m.	60 p.p.m.	80 p.p.m.	100 p.p.m.	
0	2	2	1.5	2	1	--------	1.7	0	1	0	1	0	0	--------	0.4
5	6	6	6	7.5	7.5	--------	6.6	5	5	5	5	5	5	--------	5.0
20	20	20	[3] 18.5	21	21	--------	20.5	20	15.5	16	15.5	17.5	17.5	--------	16.4
60	56.5	55	[3] 52.5	57	56	--------	56.1	60	52	52	52.5	[3] 55.5	51	--------	51.9
80	73	74.5	76	77.5	74.5	--------	75.1	80	73	72	[3] 68.5	72	72	--------	72.2
100	------	------	------	------	------	95 95.5 95.5	95.3	100	------	------	------	------	------	90.5 90.5 90	90.3

[1] Added to solution as A.C.S. NaCl or KCl in calculated amounts to give indicated oxide concentrations.
[2] In solution as A.C.S. $CaCl_2$, "low-alkali" $CaCO_3$ was dissolved with concentrated HCl, care being taken to avoid excess HCl.
[3] Not included in average.

only, this is obviously the simpler procedure. In the Public Roads laboratory, however—and it is believed the same condition will be true in most other laboratories—it is necessary to analyze a variety of materials such as soil or cement extracts with water, water analysis, etc., which require the pure NaCl–KCl standard solutions. Thus, these standard solutions are readily available. Inasmuch as the method for determining the alkalies in portland cement by flame photometry already requires a special set of standard solutions (*1*), it was considered advisable to limit the number of different types of standard solutions. In addition, once the correction curve is obtained for a particular instrument, there is no need of repeating this work unless a change is made in the reagents or in the instrument. Thus, while the use of a correction curve may seem to be at first glance an unnecessary step, for practical laboratory applications it is convenient and economical.

Since the alkali impurities in the reagents used also must be accounted for, the data obtained with the correction solutions alone cannot be used directly as the basis of the correction curve. Blank determinations were made using the method as shown. These gave apparent values of the sodium and potassium concentrations which include the effects of the 1,700 p.p.m. of CaO, in addition to the alkali impurities. Thus it is necessary to subtract from the blank determinations the effect of the calcium ion for zero concentration of the alkali. The resulting value represents the alkali impurities in the reagents, and an overall effect is calculated by adding the value of the blank to each of the values obtained with the correction solution. A curve is then constructed using these apparent values as abscissas and the corrected or true values as ordinates.

For example, the correction curves used for the determinations reported in this article were constructed as follows: The averages for each concentration shown in table 5 are equivalent to the data normally obtained with the correction solutions. Blank determinations showed an average apparent value of 2.5 p.p.m. for Na_2O and 2.0 p.p.m. for K_2O. Table 5 showed the effect of the calcium ion alone (zero concentration of alkali) to be +1.7 p.p.m. for the sodium determination, and +0.4 p.p.m. for the potassium determination. The respective differences, 0.8 p.p.m. for sodium and 1.6 p.p.m. for potassium, represent the concentration of the alkali resulting from the reagent impurities. The combined effect is obtained by adding these values to the respective average values shown in table 5. These data, given in table 6, were then used to plot the correction curves shown in figure 2.

Table 6.—*Combined effect of 1,700 p.p.m. calcium oxide and reagent impurities on the alkali determinations with the flame photometer*

[parts per million]

Na_2O added (1)	Na_2O determined[2]	K_2O added[1]	K_2O determined[2]
0	2.5	0	2.0
5	7.4	5	6.6
20	21.3	20	18.0
60	56.9	60	53.5
80	75.9	80	73.8
100	96.1	100	91.9

[1] True concentration.
[2] Apparent concentration.

Comparison with Gravimetric Method

Table 7 shows a comparison between results obtained by an accepted gravimetric procedure and by the flame-photometer method described in this article. A variety of silicates were analyzed, including a large number of soils, in order to obtain as wide a range of alkali content as possible. For most samples there is very good agreement between the flame-photometer and the gravimetric results. For Na_2O, out of 24 samples only 3 showed differences in percentage greater than 0.10. For K_2O, 9 samples showed differences in percentage greater than 0.10, but only 1 of these differences was greater than 0.15. The average difference in percentage for sodium was 0.054 while that for potassium was 0.076. Since in most cases the gravimetric result represents only one determination, there is no assurance that this result is more accurate than that obtained with the flame photometer.

While data on the reproducibility of gravimetric results are limited, table 8 shows a comparison of duplicate gravimetric results for a few materials. These analyses were made in a routine manner, employing the usual technique with the average amount of care. It is noted that the differences between the results of duplicate tests by the gravimetric method are of the same order as differences between results of the gravimetric and flame-photometer methods.

The differences between gravimetric and flame-photometer test results found in this

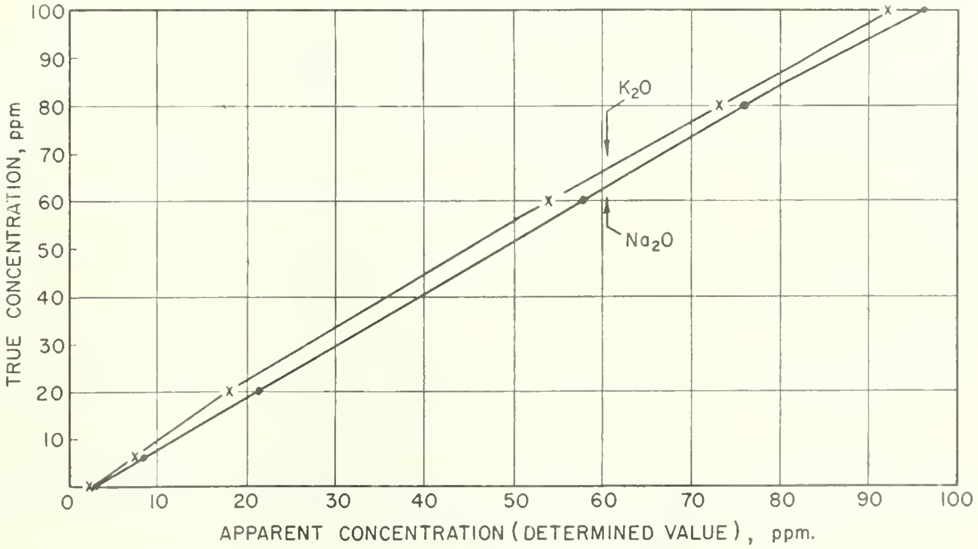

Figure 2.—Correction curves for the combined effect of 1,700 p.p.m. calcium oxide and the reagent blank on the sodium oxide and potassium oxide determinations with the flame photometer.

paper compare favorably with differences reported for other applications of the flame photometer. Barnes and others (*2*) recently described a method involving the use of an internal standard (lithium) in an instrument using filters to isolate the desired wave length. While their data for clays gave only the total alkali (Na_2O+K_2O) determined by chemical means, their differences are of the same order or slightly higher in most cases than the differences shown in this work.

For portland cement analyses the concentration of the test solution is such that 1 p.p.m. is equivalent to 0.01-percent alkali oxide. Results obtained in the Public Roads laboratory, as well as those which appear in various unpublished reports and in a report by Eubank and Bogue (*4*), show an average difference in percentage between gravimetric and flame-photometer results of from 0.01 to 0.03 for Na_2O and from 0.02 to 0.03 for K_2O. In the method for siliceous materials the concentration of the test solution is such that 1 p.p.m. is equivalent to 0.05 percent. Thus, while the actual difference reported for portland cement is less, the difference in terms of concentration of test solution in parts per million is approximately the same.

Application to Non-Portland Hydraulic Cements

An interesting observation may be made in connection with the pozzolanic cements shown in table 7. Flame-photometer results for these same cements by the tentative A.S.T.M. method for portland cements (*1*) gave, for sample 23, 0.09 percent Na_2O and 0.39 percent K_2O, and for sample 24, 0.13 percent Na_2O and 0.59 percent K_2O. These values are considerably less than those obtained by gravimetric means or by the method given here. These and similar cements have alkalies associated with the acid-insoluble residue which are not brought into solution by the tentative A.S.T.M. method for portland cements. Thus, the proposed procedure

(*Continued on page 98, third column*)

Table 7.—*Comparison of results obtained by gravimetric and flame-photometer methods*

Sample identification	Material	Amount of Na_2O			Amount of K_2O		
		Gravimetric [1]	Flame photometer	Difference [2]	Gravimetric [1]	Flame photometer	Difference [2]
		Percent	*Percent*		*Percent*	*Percent*	
1	Soil	0.15	0.14	−0.01	4.08	4.20	+0.12
2	do	.07	.08	+.01	3.76	3.87	+.11
3	do	.11	.15	+.04	.40	.34	−.06
4	do	.43	.45	+.02	1.50	1.61	+.11
5	do	.84	.82	−.02	2.22	2.18	−.04
6	do	1.03	1.01	−.02	2.14	2.21	+.07
7	do	.72	.82	+.10	2.06	2.18	+.12
8	do	.39	.42	+.03	1.57	1.56	−.01
9	do	.50	.52	+.02	1.38	1.41	+.03
10	do	.73	.77	+.04	2.88	2.94	+.06
11	do	.49	.58	+.09	1.59	1.73	+.14
12	do	.92	.93	+.01	2.00	2.24	+.24
13	do	.84	.90	+.06	2.08	2.18	+.10
14	do	.10	.10	.00	.21	.16	−.05
15	do	.15	.06	−.09	.53	.51	−.02
16	do	.31	.18	−.13	1.32	1.31	−.01
17	Slag	.19	.15	−.04	.68	.75	+.07
18	Trap rock	2.52	2.77	+.25	.76	.77	+.01
19	do	2.64	2.69	+.05	.79	.90	+.11
20	Diorite	3.90	4.08	+.18	3.08	3.23	+.15
21	Fly ash	.25	.25	.00	2.34	2.34	.00
22	do	2.74	2.69	−.05	.81	.96	+.15
23	Pozzolanic cement	.21	.19	−.02	1.23	1.24	+.01
24	do	.16	.15	−.01	.86	.89	+.03
Average		------	------	.054	------	------	.076

[1] Analyses made by J. Lawrence Smith method of fusion, with potassium being determined as K_2PtCl_6 and sodium obtained by difference from the weight of the combined sulfates.

[2] Calculated by subtracting the gravimetric from the flame photometer result.

Table 8.—*Comparison of results obtained in duplicate tests by the gravimetric determination of sodium and potassium oxide* [1]

Sample identification	Material	Amount of Na_2O found			Amount of K_2O found		
		Test 1	Test 2	Difference	Test 1	Test 2	Difference
		Percent	*Percent*		*Percent*	*Percent*	
1	Soil	0.14	0.16	0.02	4.08	4.08	0.00
3	do	.09	.14	.05	.38	.43	.05
4	do	.51	.36	.15	1.55	1.45	.10
17	Slag	.21	.17	.04	.66	.48	.18
18	Trap rock	2.55	2.50	.05	.76	.76	.00
19	do	2.65	2.64	.01	.81	.77	.04
20	Diorite	3.83	3.98	.15	3.17	3.00	.17
21	Fly ash	.23	.27	.04	2.29	2.38	.09
23	Pozzolanic cement	.21	.21	.00	1.17	1.29	.12
24	do	.17	.15	.02	.85	.88	.03
Average		----------	----------	.053	----------	----------	.078

[1] Samples analyzed by J. L. Smith method of fusion with $CaCO_3$. Potassium determined as K_2PtCl_6 and sodium determined by difference from the weight of the combined sulfates.

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Express Highways in the United States: a Bibliography.
Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF OCTOBER 31, 1950

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$11,813	$12,354	$6,333	282.7	$4,954	$2,354	155.7	$13,921	$6,249	306.1	$31,229	$14,936	744.5
Arizona	392	1,923	1,374	44.5	1,897	1,265	26.2	5,793	4,137	120.3	9,613	6,776	191.0
Arkansas	1,531	9,168	5,234	227.0	3,152	1,538	83.6	17,763	8,759	462.7	30,083	15,531	773.3
California	2,655	23,715	9,088	177.8	4,847	2,363	22.6	44,665	21,994	277.2	73,227	33,445	477.6
Colorado	1,559	4,189	2,261	74.0	2,097	1,177	61.2	15,203	8,824	316.0	21,489	12,262	451.2
Connecticut	1,373	10,322	4,752	23.4	2,212	1,274	2.4	5,379	3,080	10.2	17,913	9,106	36.0
Delaware	1,345	618	318	21.3	1,409	703	4.5	5,471	2,620	41.6	7,498	3,641	67.4
Florida	2,366	14,424	7,325	387.0	8,936	4,422	225.1	12,901	6,452	256.3	36,261	18,199	868.4
Georgia	574	17,922	9,173	398.2	9,971	4,035	115.5	27,933	13,778	664.3	55,826	26,986	1,178.0
Idaho	3,707	8,959	5,642	292.2	915	561	60.9	7,058	3,835	141.4	16,932	10,038	494.5
Illinois	20,376	39,092	20,932	324.8	10,758	5,385	76.6	51,108	25,029	365.3	100,958	51,346	766.7
Indiana	6,648	32,943	16,004	165.3	6,142	3,655	24.5	16,009	7,948	94.1	55,094	27,607	283.9
Iowa	2,951	8,903	3,409	222.7	3,349	1,182	110.9	17,455	8,254	572.0	29,707	12,845	905.6
Kansas	3,434	9,520	3,903	968.7	3,700	1,878	449.5	12,762	6,385	620.1	25,982	12,166	2,038.3
Kentucky	1,491	11,512	5,565	103.7	6,034	2,978	107.9	17,675	8,703	321.1	35,221	17,246	532.7
Louisiana	3,294	20,827	9,227	165.7	5,659	2,594	60.9	20,296	10,744	261.4	46,782	22,565	488.0
Maine	1,290	7,302	3,866	84.2	789	398	11.1	6,348	3,359	61.1	14,439	7,623	156.4
Maryland	1,313	6,714	3,192	18.7	2,889	1,165	15.5	13,234	6,175	48.1	22,837	10,532	82.3
Massachusetts	2,476	3,390	896	.7	7,018	3,602	6.0	67,871	33,078	69.7	78,279	37,576	76.4
Michigan	2,431	17,043	8,775	454.1	7,484	3,922	134.9	41,014	16,889	409.7	65,541	29,586	998.7
Minnesota	1,757	7,329	4,361	752.6	1,935	954	215.5	22,180	12,009	677.9	31,444	17,324	1,646.0
Mississippi	3,882	14,804	7,791	529.3	3,828	1,839	127.2	6,932	3,546	237.8	25,564	13,176	894.3
Missouri	5,226	29,582	16,228	871.3	6,806	2,898	214.8	30,020	14,561	481.3	66,408	33,687	1,567.4
Montana	4,572	14,853	7,673	505.1	2,301	1,389	13.4	10,577	6,418	296.7	27,731	15,480	815.2
Nebraska	4,505	16,152	8,446	533.0	4,531	2,048	98.9	10,388	5,648	285.6	31,071	16,142	917.5
Nevada	1,644	3,423	2,819	117.3	925	765	29.6	3,868	3,184	105.1	8,216	6,768	252.0
New Hampshire	1,727	3,404	1,880	28.4	1,065	486	7.7	4,350	2,081	37.6	8,819	4,447	73.7
New Jersey	2,526	3,288	1,532	6.5	2,970	1,331	5.1	18,087	8,578	27.6	24,345	11,441	39.2
New Mexico	987	3,754	2,398	115.1	5,207	3,331	143.6	6,465	4,226	185.6	15,426	9,955	444.3
New York	24,816	63,536	31,347	180.0	29,461	11,369	69.5	103,468	49,045	187.5	196,465	91,761	437.0
North Carolina	991	18,123	8,686	458.4	5,072	2,485	88.0	23,506	11,116	583.3	46,701	22,287	1,129.7
North Dakota	2,224	7,642	3,949	1,052.1	3,694	1,692	323.3	4,917	2,456	519.9	16,253	8,097	1,895.3
Ohio	8,500	26,531	12,913	281.5	20,677	11,482	156.5	53,280	25,302	280.3	100,488	49,697	718.3
Oklahoma	2,415	10,431	6,088	96.4	10,015	5,266	199.4	18,733	8,734	451.1	39,179	20,088	746.9
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South Dakota	1,028	7,153	4,051	673.3	1,747	1,028	162.1	10,621	6,451	789.1	19,521	11,530	1,624.5
Tennessee	1,116	12,713	6,162	268.0	6,493	3,030	105.0	18,594	8,485	362.9	37,800	17,677	735.9
Texas	2,175	3,269	1,678	216.0	12,964	6,918	318.7	48,473	22,707	1,219.2	64,706	31,303	1,753.9
Utah	1,417	3,946	2,888	92.3	675	500	24.3	5,968	4,358	183.4	10,589	7,746	300.0
Vermont	933	2,284	1,268	38.4	581	292	6.4	5,575	2,609	51.6	8,440	4,169	96.4
Virginia	4,272	21,455	10,628	488.1	6,793	3,340	186.2	13,253	6,477	194.5	41,501	20,445	868.8
Washington	402	11,975	4,941	74.8	2,714	1,327	36.8	18,652	9,030	171.7	33,341	15,298	283.3
West Virginia	722	16,952	7,084	135.0	2,330	1,193	45.0	8,950	4,543	77.7	28,232	12,820	257.7
Wisconsin	7,856	17,556	9,234	262.9	1,923	983	91.1	16,278	8,204	384.9	35,757	18,421	738.9
Wyoming	399	1,846	1,223	34.5	939	610	12.0	5,596	3,404	163.7	8,381	5,237	210.2
Hawaii	665	8,228	3,648	19.9	3,158	1,417	7.6	3,342	1,642	15.8	14,728	6,707	43.3
District of Columbia	1,487	4,150	2,075	4.1	899	662	1.8	1,240	619	1.8	6,289	3,356	7.7
Puerto Rico	848	13,157	6,023	66.2	2,077	870	6.1	10,024	4,220	37.7	25,258	11,113	110.0
TOTAL	169,247	645,578	323,098	12,694.3	251,800	123,960	4,601.6	1,006,112	497,911	14,049.7	1,903,490	944,969	31,345.6

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VOL. 26, NO. 6

FEBRUARY 1951

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Charts for the graphic solution of intersection capacity problems are presented in this issue

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The printing of this publication has been approved by the Director of the Bureau of the Budget January 7, 1949.

BUREAU OF PUBLIC ROADS
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Design Capacity Charts for Signalized Street and Highway Intersections

By J. E. LEISCH,[1] Highway Engineer,
Urban Highway Branch,
Bureau of Public Roads

Introduction

THE recently published *Highway Capacity Manual*[2] has furnished highway and traffic engineers with much-needed data for the design and improvement of streets and highways. This up-to-date information, which is largely based on field observations rather than on theory, should be applied to all design problems to assure the provision of appropriate and adequate highway facilities. The capacity of most streets or urban highways, except those in the freeway class, is determined largely by the volumes of traffic that can be handled at the intersections. Part V of the Manual presents a method of analysis for the calculation of capacity at signalized intersections under various conditions. Although this method is quite complete and is concisely stated in the Manual, its application, with the numerous steps necessary to obtain the result, is apt to become involved for complicated problems.

A graphic analysis method, based on Part V of the Manual, is presented here to facilitate the determination of capacities of signalized intersections. The value of charts is recognized in all fields of engineering, and their application is well demonstrated as a short cut for the use of cumbersome formulas and detailed calculations. Such charts are of considerable value in visual demonstration of the effect of each variable included. The design capacity charts developed here for the analysis of various types of signal-controlled intersections on two-way streets, one-way streets, and expressways are presented with problem examples to demonstrate their use.

A highway intersection, like any other structure, must be designed for the loads it is required to carry. In the case of a signalized intersection the loads are established by the volume, density, composition, and distribution of traffic using the intersecting facilities. The geometric features of design are determined by these traffic loads and their relation to physical characteristics and economic considerations of the site or locality, and to the type of facilities involved.

The rational determination of highway capacity is now recognized as an important part of highway planning and design. The recently published **Highway Capacity Manual** *provided, for the first time, a complete, practical method for the calculation of intersection capacities. The method developed was an arithmetic process, and for those who prefer a graphic procedure the charts presented in this article were devised. In addition to their convenience, the charts are of value in visual demonstration of the effects of the many variables involved.*

Data Needed for Capacity Analysis

Knowing the design traffic loads and the physical and economic limitations, an intersection design can be made in one of two manners: (1) a geometric layout can be prepared and by capacity analysis checked for its suitability to carry the anticipated traffic; or (2) the design traffic loads can be used in capacity analyses to determine the necessary control dimensions and operating conditions. In either event a capacity analysis is required, for which proper data are essential. Often only the average daily traffic on each intersecting street is available. This can serve as a general guide but may be of little direct value in the intersection design. For proper capacity analyses the following information is needed:

Traffic volume and distribution.—Directional design volumes of traffic on each approach to the intersection, with breakdown as to through, left-turning, and right-turning movements, together with factors for anticipated future traffic increases. In urban and suburban areas, data are needed for simultaneous movements during both the morning and evening peaks to determine the critical conditions.

Trucks and busses.—Classification of these traffic volumes to show percentage of trucks and busses.

Bus stops.—Near-side or far-side location of bus stops and approximate number of busses using them during the peak hours.

Type of area.—Classification of intersection site conditions as downtown, intermediate, or outlying area.

Traffic signals.—Type and over-all control conditions of traffic signals used.

Miscellaneous.—Requirements for parking and for pedestrians, space limitations of pavement and right-of-way areas, and other physical controls.

Development of Capacity Charts

The design capacity charts are based on the information contained in part V, Signalized Intersections, of the Manual, particularly the data given in figures 24 and 26 (pp. 79, 84), and the section on adjustments for specific conditions (pp. 87–91). Any one intersection may have as many as eight conditions for which adjustments must be made in order to determine its capacity.

The charts presented here incorporate all of these adjustments, so that for any known condition the intersection capacity can be obtained directly without reference to the various Manual adjustment values. In constructing the charts all of the adjustments are precisely accounted for and no short-cuts or approximations are made. The results obtained by the use of these charts are the same as those from the long-hand method in the Manual. There is one added adjustment to the Manual data, as explained later, in the form of a factor to obtain "design" capacity.

The material on capacity of signalized intersections could have been combined on a few charts to cover all conditions for the various types of facilities, but such charts would be unduly complex. Accordingly, it was decided to prepare a larger number of charts each containing the data for a selected condition. These charts are individually explained in the following discussion, and examples of their proper use are given in each case.

General Terms Used

In order to simplify the terms on the charts and in the examples, a system of symbols was adopted for the variable conditions that affect capacity, as follows:

$W/2$ Pavement width, in feet, of one approach to the intersection. For the two-way facilities it is normally, but not necessarily, one-half of the curb-to-curb width. For one-way facilities it is the normal curb-to-curb width, exclusive of separate turning lanes.

---

[1] Acknowledgment is made to D. W. Loutzenheiser, W. P. Walker, J. S. Biscoe, and J. A. Desch for assistance in the preparation and review of the manuscript.

[2] *Highway Capacity Manual*, by the Committee on Highway Capacity, Department of Traffic and Operations, Highway Research Board; published by the Bureau of Public Roads, 1950. Hereafter referred to in the text as the Manual.

T Trucks and busses on the one approach, expressed as a percentage of the total volume on that approach (exclusive of light delivery trucks).

R Right-turning vehicles (of all types) on the one approach, expressed as a percentage of the total volume on that approach.

L Left-turning vehicles (of all types) on the one approach, expressed as a percentage of the total volume on that approach.

B Location (or nonexistence) of a bus stop at the intersection, described as near-side stop, far-side stop, or no bus stop.

D Distance in feet that parking is prohibited in advance of the intersection, on the approach under consideration.

G/C Proportion of total time during the peak hour that the signal is green for the movement of traffic from the one approach, where *G* is the green interval in seconds and *C* is the total cycle (including the green, amber, and red intervals) in seconds.

K Design capacity of one approach, expressed in vehicles per hour.

P Possible capacity of one approach, expressed in vehicles per hour.

These terms apply to all charts. Additional terms relative to special conditions are described as they occur in the text.

Part I.—Two-Way Streets

DESIGN CAPACITY FACTORS

The basic data for intersection capacities of two-way streets, expressed in terms of average maximum volumes observed at street intersections, are shown in figure 1.[3] These data, to a large degree, represent a condition with a continual backlog of vehicles on the intersection approach so that some drivers waited through two or more cycle changes. In view of this, the Manual recommends the use of 90 percent of the values in figure 1 for what is termed practical capacity. At practical capacity, according to the Manual, traffic will pass through the intersection with few drivers having to wait longer than for the first green period. In examining these data, the Committee on Planning and Design Policies of the American Association of State Highway Officials considered that the constant factor of 90 percent for design was not representative for all cases. With recognition of the variable conditions of street width, type of area, and parking regulation, the Committee recommends that design capacity be determined by application of a factor in the range of 70 to 90 percent of the values obtained from the Manual data and shown here in figure 1. The percentages for conversion of the average values of figure 1 to design capacities, as used herein to construct the charts, are shown in table 1.

It may be noted that an 80- or 90-percent factor is used for the majority of cases. The 70-percent factor is used only for the narrower streets on which parking is permitted on both sides and only one lane is available for moving traffic in each direction.

To demonstrate the application of table 1, consider a two-way, 60-foot street, with parking on both sides, situated in an intermediate area. On this street four lanes (two in each direction) are available for movement of traffic. In figure 1, using a 60-foot street and curve for intermediate area with parking permitted, an average maximum volume of 1,440 vehicles per hour of green is given for one approach. In table 1, for intermediate area, four traffic lanes, and with two parking lanes, the adjustment factor is 80 percent. Design capacity of one approach on this street is 1,440×0.80=1,150 vehicles per hour of green.

In addition to this adjustment for design capacity, which is incorporated in the design capacity charts, it may be necessary to make a further adjustment to reflect the character and habits of drivers in a particular city or locality. Since the data in figure 1 are the average of all intersections measured, and are representative of many cities throughout the country, the maximum volumes recorded at some intersections were either above or below the average curves shown in figure 1. The relation between actual intersection capacities in a given locality and those obtained by either the Manual method or the design capacity charts can be expressed as a factor, which may be more or less than unity. This relation, referred to as the "city factor," may be determined as described later on in the section on Special Conditions, item 11, and applied as a final adjustment to the results obtained from the design capacity charts.

Table 1.—*Conversion of average maximum volumes to design capacity volumes*

Parking conditions	Percentages to convert average maximum volumes[1] to design capacities, when the number of traffic lanes[2] is—			
	Two lanes	Four lanes	Six lanes	Eight lanes
Downtown areas:				
Without parking lanes	80	80	80	80
With two parking lanes	70	80	90	......
Intermediate areas:				
Without parking lanes	80	80	90	90
With two parking lanes	70	80	90	......

[1] From figure 1. [2] Exclusive of parking lanes; percentages apply to lane widths for moving traffic in range of 10-12 feet.

Table 2.—*Factor f for conversion of design capacity to possible capacity*

Type of area	Factor *f*, when *W*/2 (in feet) is—						
	22 or less	24–26	28–30	32–34	36–38	40–42	44–48
No parking:							
Downtown	1.4	1.4	1.4	1.4	1.4	1.4	1.4
Intermediate	1.4	1.3	1.25	1.2	1.2	1.2	1.2
With parking:							
Downtown	1.6	1.5	1.4	1.3	1.25	1.2	1.2
Intermediate	1.6	1.5	1.4	1.3	1.3	1.25	1.2

RELATION OF DESIGN CAPACITY TO POSSIBLE CAPACITY

Design capacity is by no means the maximum volume that can be handled at an intersection, but is a value that preferably should be used in design to provide favorable operating conditions. Whereas design capacity represents a volume of traffic that will pass through the intersection with few drivers having to wait longer than for the first green period, possible capacity represents the maximum volume of traffic that can pass through the intersection with a continual backlog of waiting vehicles. The latter condition would be considered by most drivers as too congested, since some drivers would be obliged to wait through two or more signal cycles before proceeding through the intersection. Because of site conditions and right-of-way costs at some intersections, it may not be feasible to provide facilities based on design capacity. In such cases facilities adequate for possible capacity may be the best that can be provided.

Possible capacity, according to the Manual, is 110 percent of the average maximum volumes reported in figure 1. Design capacity, on the other hand, as used herein, is expressed as 70, 80, or 90 percent of the values in figure 1. The relation between design capacity and possible capacity

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[3] This is figure 24 (p. 79) in the Manual.

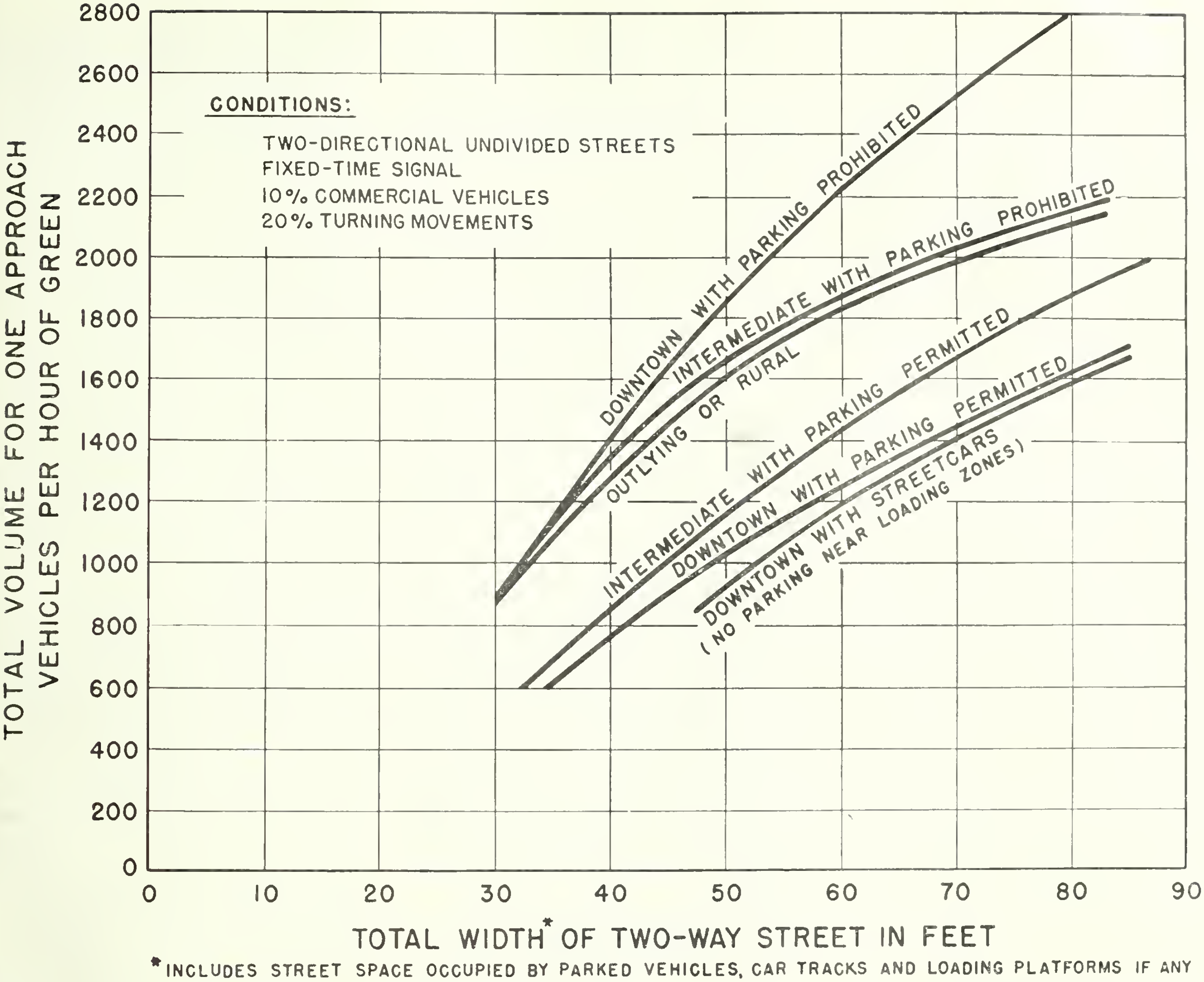

Figure 1.—Average maximum volumes at intersections on two-way streets, for different widths and by type of area and parking regulation.

is the ratio of these percentage values. This ratio provides a factor for determining possible capacity directly from the chart values for design capacity. Table 2 shows these factors, hereafter referred to as *f*, for various conditions and ranges of approach width.

The knowledge of possible capacity is needed in many instances. Although an intersection may be designed for design capacity, possible capacity will indicate the volumes that can be handled (with some congestion) on certain peak days during the year. Or, knowing possible capacity, an estimate can be made of the future point in time when no further increase in traffic can be handled.

Moreover, where separate turning lanes are provided, it may not be feasible, according to the distribution of traffic, to accommodate each movement at design capacity. One of the turning movements may have to be designed to operate at or near possible capacity. Design capacity of a separate turning lane is considered to be the same as its practical capacity. Since practical capacity on an average is 80 percent of possible capacity, the possible capacity of a separate turning lane is 1.2 times its practical capacity.

Experience has indicated that in some instances, particularly in highly developed areas, practical and economic considerations preclude widening or otherwise improving an intersection to accommodate the traffic demand without some congestion. Since the traffic for which an intersection is designed is a future volume (design volume), the intersection generally will operate satisfactorily initially even when designed for possible capacity. However, in the future when traffic builds up and equals the possible capacity, a new facility to accommodate further expansion of traffic becomes essential. Furthermore, when the traffic demand exceeds the possible capacity, operating conditions will not only be unsatisfactory but the total number of vehicles desiring to use the street cannot be served. Thus, the use of possible capacity in design, where the use of design capacity is not feasible, definitely limits the life of the facility to the date when the assumed volume is reached.

Possible capacity determined by use of the design capacity charts and table 2 is the same as that obtained by the Manual method. However, the design capacity as found by these charts, except for the narrow streets with parking on which only one lane is available for movement of traffic in each direction, is 0–10 percent lower than the practical capacity given in the Manual. The value of practical capacity, if desired, can also be obtained from the design capacity charts, as follows: (1) obtain design capacity from charts 2–6, (2) multiply by appropriate *f* in table 2, and (3) multiply the result by 0.80.

INTERSECTIONS WITH AVERAGE CONDITIONS

The first step in the development of the capacity charts was the conversion of average maximum volumes to design capacity volumes for average conditions. This was accomplished by application of the percentages in table 1 to the curves of figure 1, resulting in curves I–IV in the upper part of chart 1.[4] These curves serve as bases for the construction of other charts for two-way streets. They give design capacity in vehicles per hour of green (horizontal scale not shown). To convert such values into form for direct design use, the G/C ratio curves are plotted in the lower part of the chart. Using these, values can be read on the right vertical scale as design capacity in vehicles per hour.

Chart 1 is applicable only to average conditions, but is convenient for use in advance planning and in early stages of preliminary design. It can also be used to obtain an approximate value of design capacity at any intersection where all of the conditions are not precisely known but where the proportions of commercial vehicles, turning movements, etc., may be termed average. The subsequent charts include the adjustment factors applied to these average basic values, to obtain capacities for specific conditions.

Problems 1, 2, and 3 demonstrate the use of chart 1. The sequence through the chart may be as follows: enter at left with given width of approach; proceed right to appropriate curve designating the type of area and parking regulation; at this point turn at right angles and project downward to proper G/C curve; again turn at right angles and proceed right; read result, design capacity of one approach in vehicles per hour, on the right vertical scale. The chart may also be used in the reverse order, by entering at the right with a given approach volume and reading the result, width of approach required, on the upper left scale.

Problem 1

What is the design capacity of a two-way street, 66 feet wide curb-to-curb, with parking prohibited, in an intermediate area? Major intersections are signalized. Specific data regarding commercial vehicles, turning movements, etc., are not known, but conditions are assumed to be average. Half of the time during the hour can be allotted to green on this street.

Solution: Using $W/2=66/2=33$ and $G/C=0.50$, and following the arrows indicated in chart 1, it is found that design capacity $K=880$ v.p.h. in one direction. If parking were permitted, K would be 650 v.p.h.

Problem 2

A major street consisting of a narrow median and two 24-foot pavements, with no parking, in a downtown area, carries a volume of 950 v.p.h. in one direction during the peak hour. A signal is to be installed at a cross street. If conditions are assumed to be average, what should be the minimum green interval on the major street in order to accommodate 950 v.p.h., if a cycle of 70 seconds is used?

Solution: Enter chart 1 at left with $W/2=24$, proceed right to curve I, then down to lower graph until a horizontal projection of 950 v.p.h. is intersected; G/C is interpolated as 0.67. The minimum green interval with a 70-second cycle is, therefore, $70\times0.67=47$ seconds.

Problem 3

In a downtown area a two-way 58-foot street, with parking, intersects a two-way 44-foot street with no parking. The former is to accommodate a peak-hour volume of 530 v.p.h. in one direction. If conditions are assumed to be average, and a 60-second cycle is used (of which 6 seconds are allotted to amber) what should be the green interval on the 58-foot street for operation at design capacity? What would be the resultant green interval and design capacity of one approach on the 44-foot street? What would be the possible capacity of this approach?

Solution: Enter chart 1 at left with $W/2=58/2=29$, proceed to right to curve III, then down to lower graph until a horizontal projection of 530 v.p.h. is intersected; $G/C=0.57$. G on 58-foot street$=60\times0.57=34$ seconds. G on 44-foot street$=60-34-6=20$ seconds; and $G/C=20/60=0.33$.

For the design capacity of the 44-foot street, using $W/2=44/2=22$, curve I, and $G/C=0.33$, in chart 1, $K=420$ v.p.h. in one direction.

For the possible capacity of the 44-foot street, using $f=1.4$ from table 2 for a 22-foot approach in a downtown area with no parking, $P=420\times1.4=590$ v.p.h. in one direction.

INTERSECTIONS WITH PARKING PROHIBITED

Charts 2 and 3 include the adjustments for specific intersection conditions on two-way streets where parking is prohibited. The basic design capacity data are taken from curves I (downtown area) and II (intermediate area) in chart 1. The adjustments included are those enumerated in item I on page 88 of the Manual, covering proportions of trucks and busses, right turns, left turns, and type of bus stop. These and the following charts give the adjusted design capacity applicable for direct or final design use, whereas chart 1 is suitable primarily for preliminary investigation.

Chart 2, applicable to downtown areas with no parking allowed, permits graphical solution for a series of capacity adjustment factors. The basic scales at the left side and at the bottom are the same as in chart 1. The intermediate groups of curves cover the likely range in values for the capacity adjustments as enumerated in the Manual. In use, the sequence through the chart requires a right-angle change at the applicable value for each adjustment.

The T, R, and L adjustments are proportional corrections in terms of the vehicles involved, expressed as a percentage of the total. The B adjustment is a correction applied for the far-side, near-side, or no-bus-stop condition. This adjustment covers the cases where there is a normal number of busses stopping to pick up or discharge passengers, resulting in some signal cycles during which no busses utilize the bus-stop area. The B_x line applies to special cases with high bus volumes, explained later on in the section on Special Conditions.

Chart 2 will often be used by proceeding from the left-side to bottom scales, as shown by arrows. The chart can also be used in the reverse order to obtain the width of approach required to handle a given volume. Or, with a given approach volume and a given width, the necessary ratio of green time to cycle time can be determined readily. In the event that the approach width and the G/C ratio cannot be altered, but capacity must be increased, the amount of increase by either elimination of left turns or by changing the bus-stop condition, or both, can be found on the chart.

Chart 3 is similar in form and use to chart 2, but is for intermediate areas with no parking allowed. In figure 1 it may be noted that the curves for intermediate areas without parking and those for outlying areas nearly coincide. Since the deviation between the two is generally within about 2 or 3 percent, chart 3 may be used for both intermediate and outlying areas.

Charts 2 and 3 apply to intersection approaches having the conditions described on the charts. The direct use of the charts, as indicated by example arrows, however, is applicable specifically to the condition where the volume of left-turning vehicles on the approach can be handled without requiring a separate signal indication. A check for the capacity of left-turn movement should always be made when using charts 2 and 3, as explained later on in the section on Special Conditions, item 5. For simplicity in demonstration and better understanding of chart use, examples 4–8 are purposely selected so that the volume of left-turning vehicles does not exceed the capacity of the left-turn movement. The maximum volume of left-turning vehicles that can be accommodated without a separate signal indication on major streets is generally in the range of 80 to 120 v.p.h. Further examples illustrate the use of this important control.

Problem 4

What is the design capacity, in a downtown area, of one approach on a 64-foot street on which there is no parking, where the cycle is 60 seconds and green interval is 27 seconds? Other pertinent data are shown in the upper part of chart 2.

Solution: Enter chart 2 at left with $W/2=32$ and follow the arrows according to each condition; $K=780$ v.p.h. in one direction.

Problem 5

Determine the design capacity and possible capacity of one approach on a 46-foot street on which there is no parking, in a downtown area, with other conditions at the intersection as follows: $T=5\%$, $R=25\%$, $L=10\%$, B=near-side stop, $G=36$ seconds, and $C=60$ seconds.

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[4] The illustrative figures, incorporated in the text, and the graphic analysis charts, grouped for convenience on pp. 125-139, are independently numbered as separate series.

Solution: With $W/2=23$ and $G/C=36/60=0.60$, from chart 2, $K=700$ v.p.h. In table 2, $f=1.4$; therefore $P=700\times1.4=980$ v.p.h.

Problem 6

A new two-way street on which there will be no parking, in a downtown area, is planned to cross an existing street. According to the volume on the existing street, 33 percent of the cycle time must be allotted to green on that street. Determine the needed width of pavement on the new street if the design peak-hour volume in one direction is 1,200 v.p.h., and other conditions are as follows: $T=10\%$, $R=12\%$, $L=5\%$, B=no bus stop, and C=preferably not over 70 seconds, with 6 seconds amber per cycle.

Solution: Sixty-seven percent of the cycle time is available for amber and for green on the new street. Therefore, $(G+\text{amber})\div C=0.67$, or $(G+6)\div 70=0.67$; $G=41$ seconds, and $G/C=0.59$.

Enter chart 2 at bottom with a peak-hour volume of 1,200 v.p.h. in one direction, and proceed through the chart, turning at $G/C=0.59$, B=no bus stop, $L=5\%$, $R=12\%$, and $T=10\%$; $W/2=31.5$. If 11-foot lanes are to be used, the new street should be the nearest multiple, doubled for both directions; $33\times2=66$ feet wide.

Problem 7

Determine the design capacity of an intersection approach on a street with a narrow median and two 30-foot pavements, in an intermediate area, where other conditions are as shown in the example at the top of chart 3.

Solution: Enter chart at left with $W/2=30$ and follow the arrows according to each condition; $K=650$ v.p.h. in one direction.

Problem 8

In an outlying area a two-way, 40-foot parkway is to be crossed by a new highway. On the critical approach of the existing parkway, the peak-hour traffic is 520 automobiles in one direction of which 80 turn right and 45 turn left. On the critical approach of the new highway the design volume is 1,030 v.p.h., of which 7% are trucks, and turning movements are 10% and 4% to the right and left, respectively. There will be no parking and no bus stops at the intersection on either facility. If 12-foot lanes are to be used, how many lanes are required on one approach of the new facility?

Solution: It is first necessary to determine the proportion of green time required for the existing parkway. From chart 3, using $W/2=20$, $T=0\%$, $R=80\div520=15\%$, $L=45\div520=9\%$, and no bus stops, and intersecting from a volume of 520 v.p.h., it is found that $G/C=0.43$.

Assume $C=60$ seconds and total amber period is 6 seconds. Then, G for parkway traffic is $0.43\times60=26$ seconds, and G for traffic on the new highway is $60-26-6=28$ seconds.

To determine the required width of the new facility, enter chart 3 at the bottom with a design volume of 1,030 v.p.h. and proceed up and to the left using $G/C=28/60=0.47$, B=no bus stop, $L=4\%$, $R=10\%$, and $T=7\%$; $W/2=37$ feet. At least three 12-foot lanes, therefore, are required on the new highway in each direction of travel.

INTERSECTIONS WITH PARKING PERMITTED

Charts 4 (for downtown areas) and 6 (for intermediate areas) include the adjustments for specific intersection conditions on two-way streets with parking permitted.[5] Basic data are from curves III and IV of chart 1. The adjustments included are those enumerated in item I on pages 88 and 89 of the Manual. Charts 4 and 6 are similar to charts 2 and 3 in form and use, except that a new factor Z, for correction for bus stops, is introduced.

Chart 5 is a supplement to charts 4 and 6 to derive the adjustment factor for the combined correction for bus-stop condition and parking restriction. This correction is determined separately for near-side, far-side, and no-bus-stop condition. In chart 5A, for the near-side bus-stop condition, Z is determined directly from the $R+L$ values. In chart 5B, for the far-side bus-stop condition, Z is determined from D, G, and $R+L$ values, used jointly. In chart 5C, for the condition with no bus stops, the same three values are used jointly in a different relation. On determination of the Z value from chart 5 for the proper condition, use is then made of charts 4 and 6 in the manner previously described for charts 2 and 3.

As in the case of charts 2 and 3, the direct use of charts 4 and 6 applies to the condition where the volume of left-turning vehicles on the approach can be accommodated without requiring a separate signal indication. For simplicity in presentation, examples 9–13 were selected so that this condition is satisfied, although in actual practice a check for the capacity of left-turn movement should always be made when using charts 4 and 6, as later explained in the section on Special Conditions, item 5.

Problem 9

Determine the design capacity of one approach on a two-way, 84-foot street, with parking, in a downtown area, on which other conditions at the intersection are as listed at the top of chart 4.

Solution: Since the bus stop is on the far side, chart 5B is used: for $D=155$ feet, $G=34$ seconds, and $R+L=22\%$, $Z=17.5$. Using this and the other conditions listed, the arrows in chart 4 indicate a design capacity K of 950 v.p.h. on the one approach.

Problem 10

If all of the conditions in problem 9 remain the same except that the bus stop is placed on the near side, and there is no parking restriction on the far side, what will be the design capacity?

Solution: In this case chart 5A is used first to obtain (with $R+L=22\%$) a value of $Z=5.5$. Then, using chart 4, $K=860$ v.p.h. Shifting the bus stop from the far to the near side would decrease the capacity by 90 v.p.h.

Problem 11

If, in problem 10, the 155-foot parking restriction in advance of the intersection is retained, and all of the conditions remain the same except that the bus stop is completely removed, what will be the design capacity?

Solution: Chart 5C must be used, from which a value of $Z=12$ is obtained. Then, using chart 4, $K=910$ v.p.h. This is 50 v.p.h. more than with the bus stop on the near side (problem 10), but 40 v.p.h. less than with the bus stop on the far side (problem 9). The reason for the latter difference is that the bus-stop area on the far side (in problem 9) is used to some extent when no bus is standing, by vehicles proceeding through the intersection.

Problem 12

If the one approach of a two-way street with parking, in an intermediate area, is 32 feet wide, what is the design capacity when other controlling conditions are as listed at the top of chart 6?

Solution: Using chart 5A, with $R+L=29\%$, $Z=6$. Then, with the other conditions as listed, the arrows on chart 6 indicate a design capacity K of 770 v.p.h. on the one approach.

Problem 13

As shown in figure 2, an east-west, two-way street in an intermediate area is 52 feet wide and has parking permitted on the north side only. Two lanes are available for moving traffic on both the west and east approaches. The critical condition on the west approach occurs during the evening peak hour and on the east approach during the morning peak hour. What are the design and possible capacities of the two approaches under the conditions indicated? To what extent can capacity be increased on the east approach by eliminating parking for a half-block length ($D=170$ feet or more) in advance of the intersection?

Solution: For the west approach, since there is no parking, chart 3 is applicable and, with the conditions given in figure 2, $K=640$ v.p.h.

In table 2, for an approach width of 21 feet, no parking, in an intermediate area, $f=1.4$; then $P=640\times1.4=895$ v.p.h.

For the east approach, since there is parking, charts 5 and 6 are applicable. A value of $Z=-6$ is obtained first from chart 5C with $D=20$ feet or less and with $R+L=$

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[5] In this article, as in the Manual, parking parallel to the curb is the only type considered. Diagonal parking would obviously have a much different effect on traffic flow.

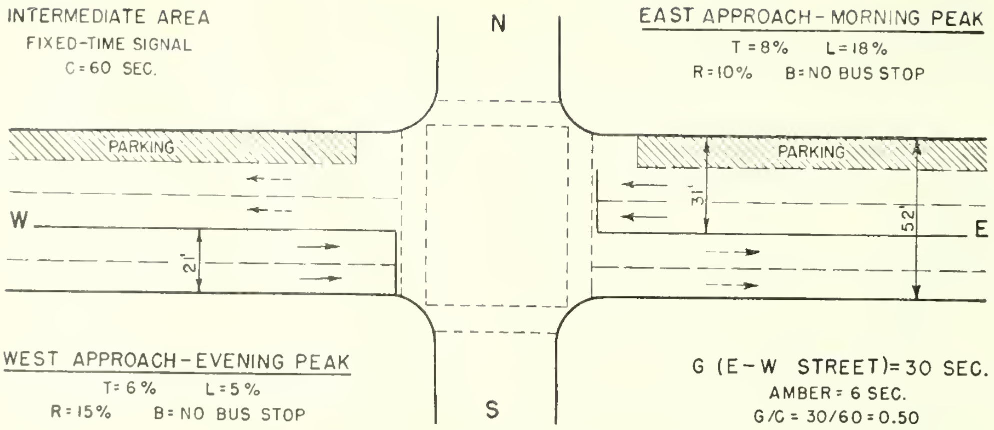

Figure 2.—Illustrative problem 13.

28%. (Since D is 20 feet or less, enter chart 5C with $D=20$ and proceed right along the upper scale to $R+L$.) Then, in chart 6, using $W/2=31$ and the other conditions shown in figure 2, $K=525$ v.p.h.

In table 2, for an approach width of 31 feet, with parking, in an intermediate area, $f=1.35$; $P=525\times1.35=710$ v.p.h.

If parking is eliminated on the east approach for a distance of 170 feet in advance of the cross walk, chart 5C is used with $D=170$ feet, $G=30$ seconds, and $R+L=28\%$; $Z=22$. Using this in chart 6, with the other conditions as before, $K=680$ v.p.h. and $P=680\times1.35=920$ v.p.h.

INTERSECTIONS WITH SEPARATE TURNING LANES AND NO SEPARATE SIGNAL INDICATION

Charts 7–9 cover intersections with separate turning lanes but with no separate signal indication. Chart 7 is used where there is a right-turn lane, chart 8 where there is a left-turn lane, and chart 9 where there are both right- and left-turn lanes.

These three charts incorporate the adjustments enumerated in item II on page 89 of the Manual. They provide graphic solutions for the design capacity of the separate turning lane and procedures for determination of the design capacity of one approach. Since the lengths of turning lanes are essential dimensions in the determination of capacity, the charts include means for determining such required lengths.

With Right-Turn Lane

Chart 7 gives the design capacity and required length of the separate right-turn lane, as well as instructions for obtaining the (total) capacity of the approach, when traffic in all lanes on the approach is permitted to move simultaneously on a common green indication. The following additional terms are introduced in chart 7.

D_2 Effective length of right-turn lane, in feet, for the storage of turning vehicles, exclusive of cross walk and taper.

V_2 Volume of traffic turning right on one approach, in vehicles per hour.

T_2 Trucks and busses turning right, expressed as a percentage of the total right-turn volume V_2 on one approach.

M_2 Design capacity of combined through and left-turn movement, exclusive of the movement on a separate right-turn lane; for use in chart 7 to obtain the (total) capacity of the approach.

K_2 Design capacity of the added lane for right-turn movement, in vehicles per hour.

The capacity of a right-turn lane is largely dependent on the proportion of truck traffic T_2 and the G/C ratio available for movement of traffic in the lane. Charts 7A and 7B show design capacity in terms of these factors. Right-turn lane capacity is also dependent on the radius of the turn, the amount of pedestrian interference, and the length of lane provided. From available data, distinction has been made for two general conditions in regard to radius and pedestrians. Chart 7A represents average curb return (corner radius at edge of pavement) and pedestrian interference, based on an average flow of 600 vehicles per hour of green. For better conditions with an adequate curb return and little or no pedestrian interference, chart 7B is constructed on the basis of an average of 800 vehicles per hour of green. Design capacity as expressed in the charts is 90 percent of these average values. In each case the intersection diagrams above the charts are indicative of the conditions represented.

Another control in capacity of an added turning lane is the length of that lane. If not long enough to store the vehicles that can make the turn on the proper green interval, the capacity otherwise possible cannot be attained. The Manual adjustments, in item II-3 on page 89, include a volume check in terms of D_2, the length of added turning lane. Chart 7C gives the solution for this length of added lane required to accommodate given volumes for different signal timings. In this form the length can be determined both for capacity volumes and for known smaller turning volumes for a specific condition. Since control values are in terms of passenger vehicles only, the adjustment for percentage of trucks and busses is included.

The required length of added right-turn lane is determined as the distance needed to store the average number of turning vehicles that will accumulate per cycle during the red and amber signals, recognizing the maximum that actually can move on the green signal. A length of 25 feet is used for each passenger vehicle, and 40 feet for each truck or bus.

The sloping lines in the lower part of chart 7C are curved to terminate at the left in logical minimum design values, according to the proportion of trucks and busses in the total traffic:

	feet
None	50
10–20 percent	65
30 percent or more	80

The minimum length of turning lane applies to the full width of turning lane. This full length is not available for use unless preceded by a taper of suitable length. While a taper length of 70 to 100 feet may be considered desirable for normal street conditions, a taper of at least 50 feet (about 5 feet of length per foot of turning lane width) should be provided. This taper is in addition to the minimum length of turning lane shown in chart 7C.

Included on chart 7 are instructions for determining the design capacity of one approach as the sum of separate values for the capacity of the through and left-turn lanes and that of the right-turn lane. Since the through-lane capacity is dependent upon the turning movements involved, the capacity for the whole is determined for a particular condition of turning movements. This value differs from a capacity sum of left plus through plus right in that it includes adjustment for any one of the three parts being at capacity while the other two are below capacity.

Since R and L are defined as percentages of the total approach volume, a simple proportion calculation is needed in step 3 of the instructions to find a right-turn volume V_2 on the arithmetic basis of through plus left. When V_2 is less than the right-turn lane capacity K_2, the design capacity is the sum of the values found in steps 1 and 3. If V_2 exceeds K_2, it is necessary to determine an adjusted volume for the combined through and left movement M'_2 based on the controlling value of the right-turn lane capacity as indicated in step 5. The formula shown is derived from the formula in step 3, with K_2 substituted for V_2. The design capacity of the approach, then, is the sum of the adjusted through-plus-left volume and the right-turn lane capacity.

The steps enumerated in chart 7 for design capacity of one approach are for the condition where each of the movements involved—left, through, and right—does not exceed the design capacity. Actually, a perfect balance between these three movements will seldom, if ever, exist. It is likely that one or both of the turning movements may exceed the design capacity value and may have to operate at or near possible capacity. In such cases, the value of K_2 in step 5 may be used as greater than design capacity, up to a maximum of the possible capacity. (This also applies to charts 8, 9, and 10.)

With Left-Turn Lane

Chart 8 gives the design capacity and the length of the separate left-turn lane, as well as instructions for obtaining the (total) capacity of the approach, and is similar in form to chart 7. Additional terms introduced are as follows:

D_3 Effective length of left-turn lane, in feet, for the storage of turning vehicles, exclusive of cross walk and taper.

V_3 Volume of traffic turning left on one approach, in vehicles per hour.

T_3 Trucks and busses turning left, expressed as a percentage of the total left-turn volume V_3, on one approach.

V_0 Volume of through traffic on the opposite approach, in vehicles per hour, that is in direct conflict, during the same period of time, with the left-turning movement on the approach in question.

T_0 Trucks and busses, expressed as a percentage of the total through volume V_0 on the opposite approach.

M_3 Design capacity of a combined through and right-turn movement, exclusive of the movement on a separate left-turn lane; for use in chart 8 in obtaining the (total) capacity of the approach.

K_3 Design capacity of the added lane for left-turn movement, in vehicles per hour.

The capacity of a left-turn lane is determined primarily by the volume of traffic opposing the left turn during the green signal indication. Normally, on major streets in downtown areas and on wide major streets in intermediate areas, it seldom will be possible for more than two vehicles to turn left per cycle (such turns usually have to be made on the amber signal). Using design capacity of turning lanes as 80 percent of possible capacity, the design capacity of a left-turn lane is, then, 1.6 vehicles per cycle. Chart 8B gives this relation in terms of vehicles per hour as dependent upon the length of cycle.

On some streets, where the opposing through volume is relatively light, the capacity of a left-turn lane may be much greater than indicated above. For such a condition the average capacity of a left-turn lane per hour of green is estimated as the difference between 1,200 (item II-3-b, p. 89 of the Manual) and V_0, both figures expressed in terms of passenger vehicles. Design capacity is 90 percent of this difference. Chart 8A provides a solution for this condition, being the relation between V_0, G/C, and design capacity K_3. To express the capacity in terms of vehicles of all types, factors for the percentage of trucks and busses in the opposing through movement T_0 and that in the left-turn movement T_3 are applied. To determine the capacity of a left-turn lane, K_3 should be found on both charts 8A and 8B, and the larger of the two values used. In most cases on major streets, the values from chart 8B will govern.

Because of the interference of opposing through traffic, left-turning vehicles generally are delayed for longer periods of time than right-turning vehicles. The required green time per vehicle to make the turn is greater and the left-turn lane capacity is less than that for a right-turn lane. Moreover, when drivers not in the added left-turn lane await an opportunity to turn, those going right generally offer little interference to through traffic but those going left often block a through lane. Thus, within capacity conditions, a longer added lane for left turns is needed for a given volume than for an added lane for right turns of the same volume. Figure 8C shows required lengths of left-turn lanes based on storage space for 1.5 times the average number of turning vehicles that will accumulate per cycle. This is an assumed factor (50 percent increase over right-turn requirements).

With Both Right- and Left-Turn Lanes

Where lanes are added for both right and left turns, the determination of design capacity of an approach is made by steps as shown in chart 9. Design capacities of the added lanes, K_2 and K_3, are determined from charts 7 and 8 and, depending upon actual turning volumes involved, adjustments are made to find the proper through capacity M_1. M_1 is the design capacity of the lanes for through movement, exclusive of the movements on separate right- and left-turn lanes, for use in chart 9 to obtain the (total) capacity of the approach. Step 2 determines the through capacity from charts 2 or 3. In step 4 the through capacity, thus determined, is added to the turning volumes calculated in step 3. Step 5 adjusts for the through volume when one of the turning volumes exceeds design capacity, the basis being the same as described for chart 7. If both turning movements exceed design capacity, a separate step 5 solution should be made for each and the smaller of the two values obtained will govern the design capacity of the approach. Step 6 derives D_2 and D_3 from charts 7 and 8.

These six steps are for the determination of design capacity of one approach for the condition when no movement exceeds design capacity. Sometimes the traffic load may be such that the through movement can be accommodated at design capacity M_1 as determined in step 2, but the proportional volumes V_2 and V_3, as found in step 3, exceed the design capacities K_2 and K_3. In such a case it would be necessary to permit the turning movements to operate at above design capacity but not to exceed the possible capacity of each. Thus, the capacity of the approach may be equal to M_1+V_2 (not to exceed 1.2 K_2)$+V_3$ (not to exceed 1.2 K_3).

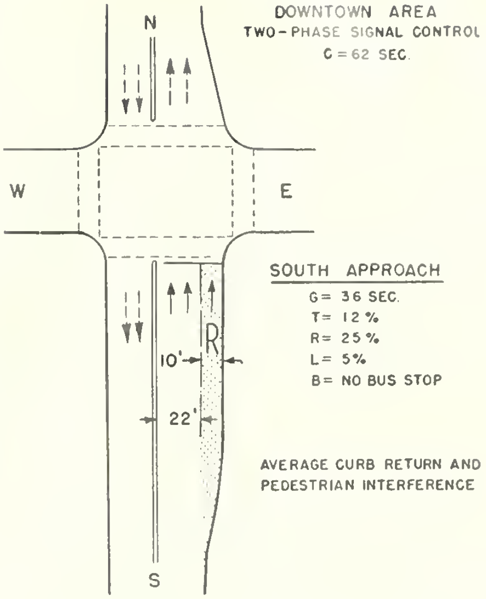

Figure 3.—Illustrative problem 14.

Problems 14–17, which follow, are illustrative of the uses of charts 7, 8, and 9.

Problem 14

What is the design capacity of the south approach for the conditions indicated in figure 3? What should be the length of the right-turn lane?

Solution: The percentage that the right-turning trucks are of the right-turn volume is not given, so T_2 is assumed to be the same as T, or 12%. Using $G/C=36/62=0.58$ and $T_2=12\%$, from chart 7A $K_2=305$ v.p.h.

From chart 2, using $W/2=22$, $T=12\%$, $R=0\%$, $L=5\%$, $B=$no bus stop, and $G/C=0.58$, the design capacity of combined through and left-turn movement M_2 (K on chart 2)$=830$ v.p.h. (see step 2 in chart 7).

On this basis, (from step 3, chart 7) the right-turn volume $V_2=(830\times 25)\div(100-25)=275$ v.p.h.

Since this is less than K_2, the design capacity of the south approach $K=830+275=1{,}105$ v.p.h. (step 4 in chart 7).

The length of right-turn lane required, D_2 from chart 7C, using $V_2=275$ v.p.h., $C=62$ seconds, and $T_2=10$ to 20%, is 160 feet.

Problem 15

Determine the length of green interval required to handle the traffic at design capacity on the east approach of the intersection shown in figure 4.

Solution: Signal timing based on a combined volume of through and right-turn movement is obtained from chart 3 using $W/2=20$, $T=(15+40)\div(175+505)=8\%$, $R=175\div930=19\%$, $L=0\%$ (since left turn is on separate lane), $B=$no bus stop, and approach volume $= 175+505 = 680$ v.p.h.; $G/C=0.57$.

Then check, in charts 8A and 8B, the capacity of the left-turn lane with this signal timing. Entering chart 8A with

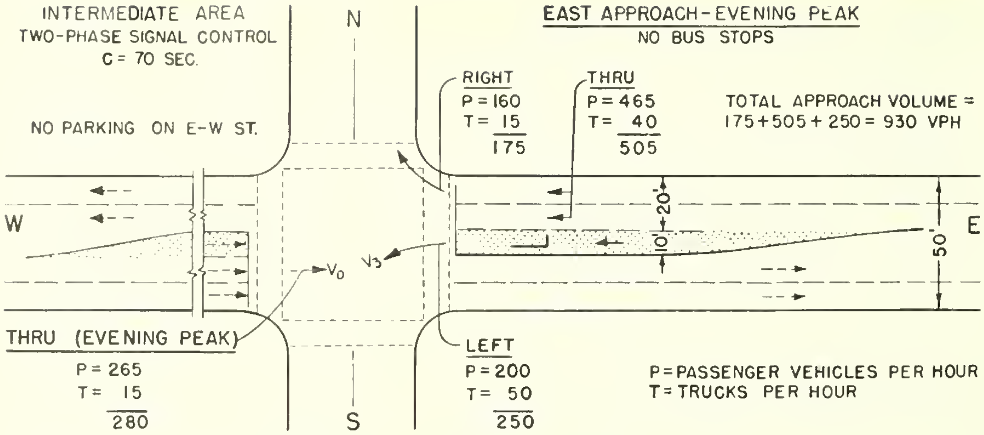

Figure 4.—Illustrative problems 15 and 16.

V_0=280 v.p.h. and proceeding to right and bottom with $T_0=15\div280=5\%$, $G/C=0.57$, and $T_3=50\div250=20\%$ (see arrows), the design capacity of the left-turn lane is $K_3=280$ v.p.h. Chart 8A governs since in this case the value of 82 from chart 8B is much less. Thus, the indicated left-turning volume of 250 v.p.h. can be accommodated.

Therefore, for the east approach an adequate green interval $G=0.57\times70=40$ seconds.

The required length of the left-turn lane to handle a volume of 250 v.p.h. is obtained from chart 8C. Using $C=70$ seconds and $T_3=20\%$, $D_3=205$ feet.

Problem 16

If in problem 15 the G/C ratio of 0.57 is retained and other conditions (fig. 4) remain the same except that parking is permitted and a bus stop is placed on the near side, what will be the design capacity and possible capacity of the east approach?

Solution: The capacity of the combined through and right-turn movement is obtained from charts 5 and 6. Using chart 5A first, with $R+L=19+0=19\%$ (L used as 0% since it is on separate lane), $Z=5$ is obtained. Chart 6 is then used according to instructions in step 2 of chart 8. With $W/2=20$, $T=8\%$, $R=19\%$, $L=0\%$, $Z=5$, and $G/C=0.57$, design capacity (exclusive of left turn) M_3 is found to be 380 v.p.h.

Left-turn movement L, according to the traffic distribution shown in figure 4, is $250\div930=27\%$. The left-turn volume V_3 on the basis of M_3 is $(380\times27)\div(100-27)=140$ v.p.h. (see step 3 in chart 8).

Design capacity of east approach=380+140=520 v.p.h. With no parking and no bus stop (problem 15), design capacity is the sum of volumes indicated in figure 4, or 175+505+250=930 v.p.h.

In table 2, for $W/2=20$, with parking, in an intermediate area, $f=1.6$. Possible capacity of combined through and right-turn movement $= M_3\times1.6 = 380\times1.6 = 610$ v.p.h. Corresponding left-turn volume (from the formula in step 3, chart 8) $=(610\times27)\div(100-27)=225$ v.p.h., which can be handled since $K_3=280$ v.p.h. (as determined in problem 15).

Possible capacity of east approach=610+225=835 v.p.h.

Problem 17

What is the design capacity of the east approach shown in figure 5? A large lumber mill to the north on the cross road accounts for the sizable proportion of vehicles and the high percentage of trucks turning right.

Solution: From chart 7B, $K_2=280$ v.p.h. Since chart 8B generally governs the capacity of the left turn on major multilane streets, it is used initially, obtaining $K_3=95$ v.p.h.

According to step 2 in chart 9, chart 3 is used, with $W/2=22$, $T=6\%$, $R=0\%$, $L=0\%$, B=far-side bus stop, and $G/C=30/60=0.50$; M_1 is 610 v.p.h.

On this basis, $V_2=(610\times32)\div(100-32-8)=325$ v.p.h., and $V_3=(610\times8)\div(100-32-8)=80$ v.p.h. (see step 3 in chart 9). Thus V_2 is larger than K_2 and, in order that no movement shall exceed the design capacity, it is necessary to recalculate M_1 as shown in step 5 of chart 9: $M'_1=280\ (100-32-8)\div32=525$ v.p.h.; then left-turn volume $V'_3=(525\times8)\div(100-32-8)=70$ v.p.h., which can be handled since it is less than K_3.

Design capacity of the east approach (when no individual movement exceeds its design capacity) $=525+280+70=875$ v.p.h.

Required lengths of turning lanes are: from chart 7C, $D_2=165$ feet; from chart 8C, $D_3=70$ feet.

In the event that a through volume of about 610 v.p.h. (as determined initially) had to be accommodated and other conditions could not be altered, the 325 right-turning vehicles could still be handled but at near possible capacity, since possible capacity of the right-turn lane would be 1.2 K_2, or $1.2\times280=335$ v.p.h. (see note in chart 9).

The capacity of the east approach, with the through movement operating at design capacity and the right-turn movement at near possible capacity, $=M_1+V_2+V_3=610+325+80=1{,}015$ v.p.h.

INTERSECTIONS WITH SEPARATE TURNING LANES AND SEPARATE SIGNAL INDICATION

Chart 10, which is similar to charts 7, 8, and 9, gives the design capacity and the required length of right- or left-turn lane, when traffic on this lane moves on a green indication separate from that for other traffic on the approach; i. e., a right- or left-arrow indication for the turning movement.

Additional terms introduced in chart 10 are as follows:

G' Green interval, in seconds, of separate signal indication for the movement of traffic on a separate turning lane.

a Width of turning lane, in feet, for the movement of traffic on a separate signal indication.

With a separate signal indication, the right- or left-turn movement is assumed to be free from interference of other traffic streams and pedestrian movements. The average capacity of the turning lane is 800 vehicles per hour of separate green indication (item III, p. 89 of the Manual) for a lane 10 feet wide, and varies directly with the lane width. Design capacity is 90 percent of this value. Chart 10A gives a solution for this capacity in terms of G'/C and the width of lane a. Since the control values are in terms of passenger vehicles, an adjustment for the percentage of trucks and busses turning (T_2 or T_3) is included. The design capacity of the turning lane is the same whether the movement is to the right

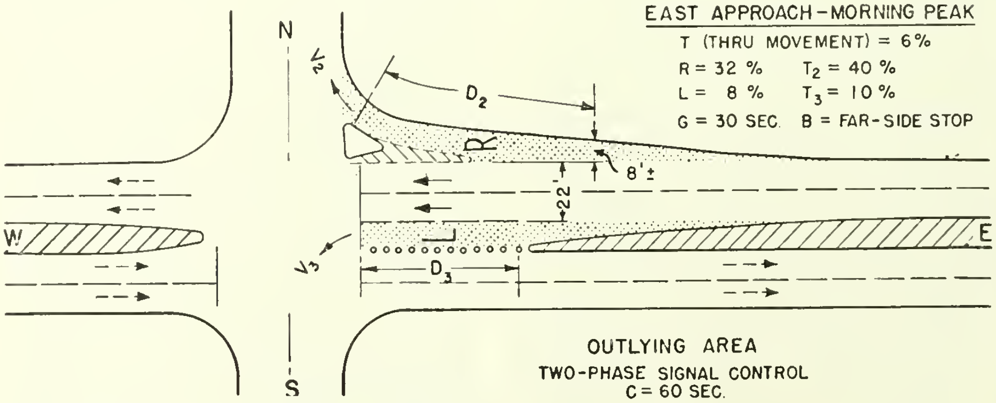

Figure 5.—Illustrative problem 17.

or to the left and whether the lane is within the normal pavement width or is an added lane.

Chart 10B provides the solution for the required length of turning lane, identical with chart 8C. In the case of separate signal indication there is no opposing traffic, as in chart 8C, but otherwise the conditions are comparable. Usually the separate signal phase is green while other through movements are stopped, and the storage lane must be long enough to prevent blocking of a through lane. This calls for a space greater than that needed to store the average number of vehicles arriving per cycle, since the number arriving on some cycle intervals will be in excess of the average. Accordingly, a length to store 1.5 times the average number of vehicles per cycle is used. This is an assumed factor (50 percent increase over normal requirements).

The procedure for determining design capacity of one approach is the same as that previously explained for charts 7 and 8. For clarity in the terms and charts, the steps are shown separately for right-turn and left-turn lanes.

Problems 18 and 19 demonstrate the use of chart 10.

Problem 18

What is the design capacity of a right-turn lane, 11 feet wide, for which a separate green indication of 20 seconds is used out of a 90-second cycle, and on which trucks and busses comprise 15 percent of the total right-turning traffic? What should be its length if a volume equivalent to design capacity is to be accommodated?

Solution: From chart 10A, using $G'/C=20/90=0.22$, $a=11$, and $T_2=15\%$ (see arrows); $K_2=165$ v.p.h. From chart 10B, using $V_2=165$ v.p.h., $C=90$, and $T_2=10$ to 20%; $D_2=175$ feet.

Problem 19

What should be the green interval for each phase on the east approach of the intersection shown in figure 6 for operation at design capacity, if a total approach volume of 650 v.p.h. is to be accommodated? What will be the green interval for the movement of traffic on the cross street?

Solution: Volume of left-turning traffic is 20 percent of 650=130 v.p.h. The volume of through and right-turn movements to be accommodated on a width of 22 feet is 650−130=520 v.p.h. The proportion of green time required for this movement, from chart 2, with $W/2=22$, $T=11\%$, $R=12\%$, $L=0\%$, B=far-side stop, and $K=520$, is $G/C=0.40$. $G=80\times0.40=32$ seconds.

The proportion of green time required for the separate phase of left-turn lane is obtained from chart 10A. Using a volume of 130 v.p.h., $T_3=15\%$, and $a=12$ feet, obtain $G'/C=0.17$. Hence $G'=80\times0.17=14$ seconds.

Green time available for movement of traffic on the cross street is 80−32−14−9 (for amber)=25 seconds.

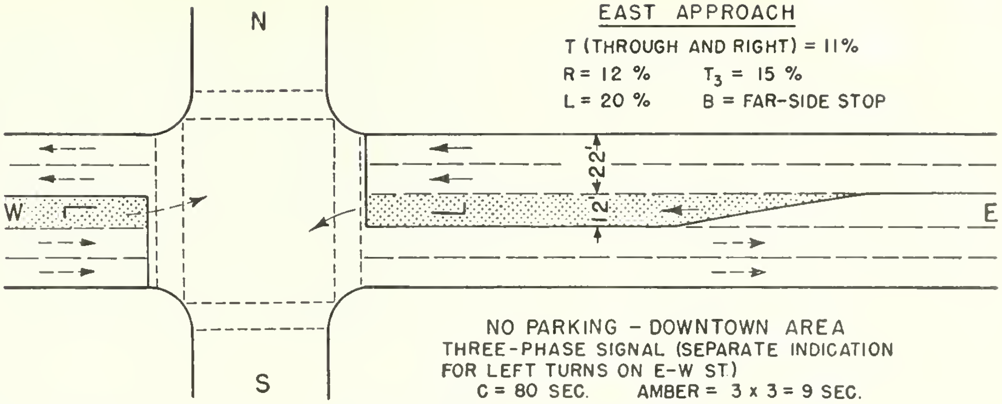
N
EAST APPROACH
T (THROUGH AND RIGHT) = 11%
R = 12 %
T_3 = 15 %
L = 20 %
B = FAR-SIDE STOP
W
22'
12'
E
NO PARKING - DOWNTOWN AREA
THREE-PHASE SIGNAL (SEPARATE INDICATION FOR LEFT TURNS ON E-W ST.)
C = 80 SEC.
AMBER = 3 x 3 = 9 SEC.
S

Figure 6.—Illustrative problem 19.

SPECIAL CONDITIONS

Charts 1–10 cover the general conditions found at four-way intersections under traffic-signal control. In addition there are numerous other conditions, the majority of which are discussed in the Manual as further adjustments. For each of these special conditions the chart procedure involves a series of steps or a special instruction to be followed.

1. *High Volume of Stopping Busses, With No Parking on Approach*

Where either a near- or far-side bus stop is provided, and where there is a high volume of stopping busses, i. e., at least one bus loading or unloading at all times (one or two stopping to load and unload per cycle in one direction of travel), use chart 2 or 3 in the normal manner, except:

(*a*) Enter chart with $W/2$ equal to actual approach width minus 12 feet.

(*b*) Use T as a percentage of trucks only, exclusive of stopping busses.

(*c*) Use line B_x for bus-stop condition.

(*d*) To the design capacity obtained from the chart, add the number of stopping busses per hour in one direction to determine total design capacity.

Problem 20

If, in problem 4 (p. 108), all of the conditions remain the same except that 90 busses stop during the peak hour on the one approach and the percentage of trucks alone is 5 percent, what will be the design capacity?

Solution: Using chart 2, with $W/2=32-12=20$, $T=5\%$, bus-stop condition line B_x, $R=7\%$, $L=15\%$, and $G/C=0.45$, a value of 510 v.p.h. is obtained. Total $K=510+90=600$ v.p.h. in one direction.

2. *High Volume of Stopping Busses, With Parking on Approach*

Where either a near-side or far-side bus stop is provided and where there is a high volume of stopping busses as described in item 1 above, use charts 4 or 6 in the normal manner, except:

(*a*) Enter chart with $W/2$ equal to actual approach width minus 6 feet.

(*b*) Use T as percentage of trucks exclusive of stopping busses.

(*c*) Use line B_y for bus-stop condition. Do not use chart 5.

(*d*) Obtain result, but add to this the number of stopping busses per hour in one direction to determine total design capacity.

Problem 21

If, in problem 9 (p. 109), all of the conditions remain the same except that 80 busses stop during the peak hour on the one approach, and the percentage of trucks alone is 4 percent, what will be the design capacity? What will be the possible capacity of this approach?

Solution: Using chart 4, with $W/2=42-6=36$, $T=4\%$, bus-stop condition line B_y, $R=15\%$, $L=7\%$, and $G/C=0.52$, a value of 700 v.p.h. is obtained. Total $K=700+80=780$ v.p.h. in one direction.

From table 2, $f=1.2$. $P=(700\times1.2)+80=920$ v.p.h. in one direction.

3. *Widened Intersections, with No Parking*

Where the pavement approach is widened in advance of the intersection for a distance in feet equal to or greater than five times the green interval in seconds ($5G$), and the same pavement widening is continued beyond the intersection for a distance in feet equal to $5G$ or more: Enter chart 2 or 3 with $W/2$ equal to total width of one approach including the widening on that approach; then use the chart in normal manner.

Problem 22

What is the design capacity of the south approach, widened through the intersection, as shown in figure 7?

Solution: Since the length of widening in advance of and beyond the intersection is greater than $5G$ ($5\times28=140$ feet), the full width of approach, $21+11=32$ feet, is used to determine the capacity.

Using chart 3, for the conditions given, design capacity of south approach=625 v.p.h.

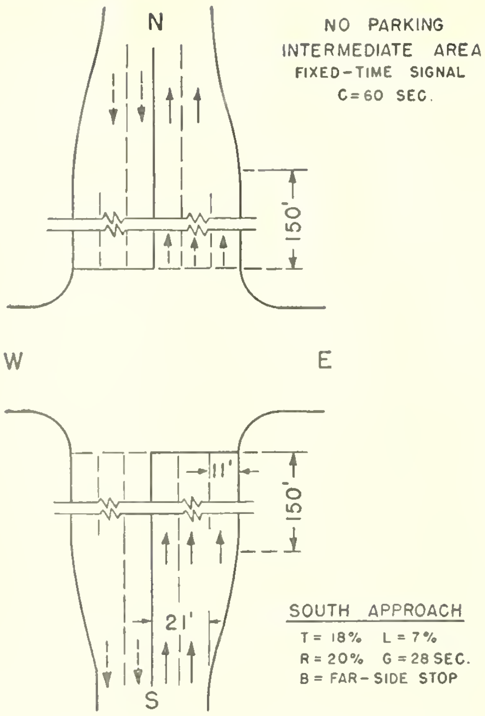

Figure 7.—Illustrative problem 22.

4. *Elimination of Parking at Intersections*

Where parking on a street is eliminated in advance of the intersection for a distance in feet equal to or greater than $5G$ and parking is also eliminated beyond the intersection for a distance equal to or greater than $5G$: Use chart 2 or 3, instead of chart 4, 5, or 6, as if there were no parking on the street.

Problem 23

To improve operation at a major intersection in a downtown area on a 42-foot street on which parking is permitted, it was decided to remove the bus stops and to prohibit parking for an effective distance in advance of and beyond this intersection. Other conditions on one approach are $T=15\%$, $R=20\%$, $L=10\%$, $G=30$ seconds, and $C=60$ seconds. Determine design capacity and possible capacity of one approach.

Solution: To be fully effective, parking must be eliminated on each side of the intersection for a distance of at least $5G$ or 150 feet.

From chart 2, using $W/2=21$ and other conditions as listed, $K=570$ v.p.h. $P=570\times 1.4$ (from table 2) $=800$ v.p.h.

5. *Check for Capacity of Left Turn*

Any intersection approach on a two-way street that does not involve a separate left-turn lane should be checked for capacity of the left-turn movement. This may be done in the same manner as for a separate left-turn lane, since the number of left-turning vehicles that can be accommodated with two-phase control, whether on a separate lane or not, is governed either by the volume of traffic opposing the left turn or by the length of cycle. Charts 8A and 8B should be used for such a check, which should be made for every intersection involving two-way streets. If the volume of left-turning vehicles exceeds the possible capacity as determined in charts 8A and 8B, serious congestion may result and the over-all capacity of the approach may be materially reduced. In such cases, the left-turn movement should be prohibited or, if feasible, accommodated on a separate signal indication.

Problem 24

Check whether the left-turn volume in problem 6 (page 109) can be handled satisfactorily.

Solution: Left-turn volume is 5% of 1,200, or 60 v.p.h. For the conditions given, it is found in chart 8B that 82 v.p.h. can be accommodated at design capacity; therefore, the solution in example 6 is satisfactory.

6. *Special Treatment of Turning Movements*

On the intersection approach of a two-way facility where the right-turn path is reasonably direct, and pedestrian interference is minor, the right-turn movement may be considered as part of the through movement; in which case, $R=0\%$ would be used in the chart solution.

On the intersection approach of a one-way facility where the turning conditions are as described above, either the right- or left-turn movement, or both, may be considered as part of the through movement. Such conditions are likely to occur at high-type, channelized intersections.

7. *Capacity Controlled by Intersection Exit*

Generally the capacity of the approaches controls the capacity of the intersection. At some locations, however, where all pavements of the intersection legs are not of the same width or where traffic backs up from an adjacent intersection, the capacity of the intersection may be dependent upon the exit lanes. The capacity of the exit pavement may be estimated as follows:

No parking on exit.—Enter chart 2 or 3 with $W/2$ equal to the width of exit pavement; proceed through chart in normal manner, but use T=percentage of trucks in through movement only, $R=0\%$, $L=0\%$, B=no bus stop (except where bus stop is on the far side, use B=far-side stop), and G/C equivalent to that used on the approach.

With parking on exit.—Enter chart 4 or 6 with $W/2$ equal to the width of exit pavement (including the parking width); proceed through chart in normal manner, but use T=percentage of trucks in through movement only, $R=0\%$, $L=0\%$, $Z=0$, and G/C equivalent to that used on the approach.

8. T *or* Y *Intersections*

The capacity of the approach on an intercepted street at T or Y intersection (east approach in figure 8) may be obtained from charts 2–6, as follows: Use the charts in normal manner except that the left-turn movement is considered to be the through movement, since it is equivalent to the through movement on a crossing (the only difference being the curved path), which makes $L=0\%$. If parking is permitted on the intercepted approach, use supplemental charts 5A and 5C, but $R+L$ is always equal to R only, because $L=0$.

Problem 25

What is the design capacity of the intercepted street (east approach), from which traffic can turn only right and left into the north-south street, for the conditions indicated in figure 8?

Solution: In chart 2, using $W/2=20$, $T=10\%$, $R=40\%$, $L=0\%$, B=no bus stop, and $G/C=0.40$, $K=460$ v.p.h. is obtained. This is the combined volume turning right and left into the north-south street.

9. *Multiple-Type Intersections*

The capacity of any form of signalized intersection, regardless of the number of approach roads and extent of channelization, can be obtained from the charts by examining each approach road separately. The design of complex intersections, particularly those requiring multiphase control, may necessitate some study and trial solutions before determining the final plan. Multiple intersections often present several possibilities in the pattern of operation and in the number and arrangement of signal phases. Such alternate arrangements are apt to result in different geometric layouts, thus affecting the size, shape, and location of islands, widths of pavements, size of storage areas, and over-all space requirements for the intersection. The geometric layout should be determined jointly with capacity analyses. Care should be taken

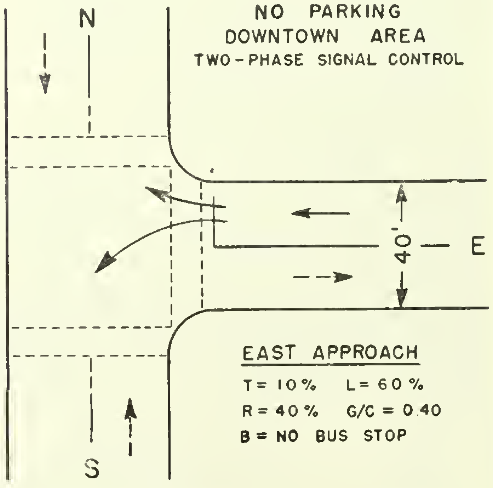

Figure 8.—Illustrative problem 25.

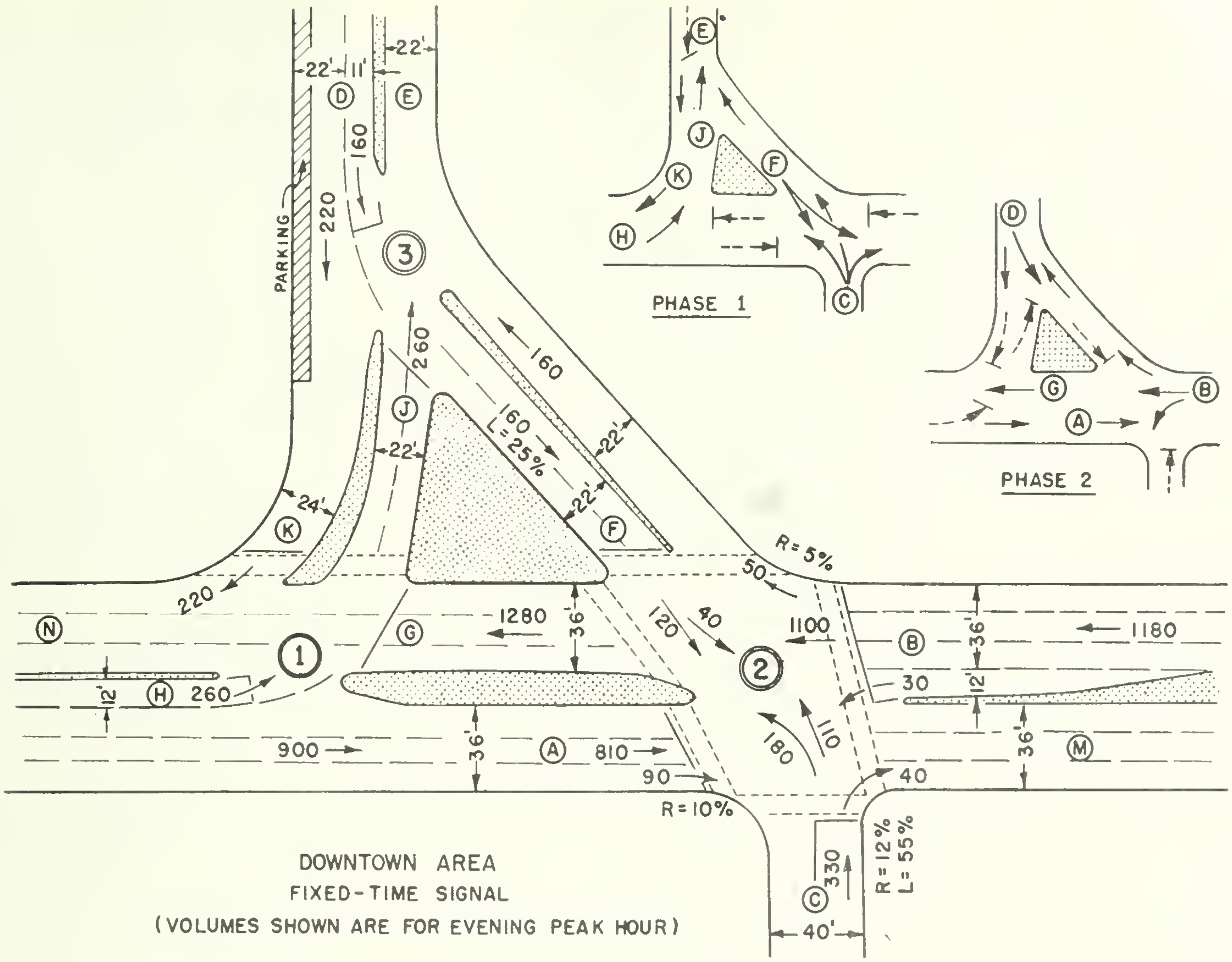

Figure 9.—Illustrative problem 26.

to check the length and width of those traffic channels where vehicles will store during certain signal phases to preclude the condition of traffic backing up from one intersection point to another. The alternate solutions will show differences in capacity, and in operational and economic advantages, from which the most feasible plan may be determined.

Problem 26

In figure 9 is shown a plan of a multiple intersection, selected from a study in which several layouts were examined. The problem is to check the arrangement for operation at or near design capacity and to determine the signal timing, using a 60-second cycle, for the two-phase control indicated at the upper right of figure 9. There are no bus stops, and T is assumed to be 15 percent on all movements.

In order that the exit lanes (due to stored vehicles) do not limit the capacity of approaches, a lagging green indication in several instances is considered necessary, as demonstrated below.

Movement H to E—phase 1

Approach H: From chart 10A, required $G'/C=0.32$, and $G=60\times0.32=19$ seconds. From chart 10B, required length of left-turn lane is 180 feet.

Approach J: To prevent an excessive number of vehicles from storing at J and thereby limiting the capacity of this movement at H, a green indication at intersection 3 to lag behind that at intersection 1 is introduced. For a distance of 140 feet from H to J and an assumed speed of 15 m.p.h. (20 feet per second), the green lag= 140/20=7 seconds. This will preclude the storage of but a few vehicles at J. Thus, for approach J, $G=19+7=25$ seconds, and $G/C=0.42$.

Approach K—phase 1

Although there is parking about 70 feet in back of the stop line, it is not likely to have any effect on the capacity of the 24-foot pavement at intersection 1 due to the variable width involved. Moreover, since all traffic turns right on a rather direct path, the movement is considered to be a through movement without turns (see Special Condition, item 6). In chart 2, for $W/2=24$, no turns, etc., and $G/C=0.32$, (same as at H), design capacity=520 v.p.h., which is more than adequate for the load.

Approach G—phase 2

Using a 3-second amber period with each phase, the available green time per cycle is $60-19-6=35$ seconds, or $G/C=0.58$. From chart 2, design capacity=1,370 v.p.h., which is adequate since the load is 1,280 v.p.h.

Approach B—phase 2

During phase 1, approach G should be left clear, or nearly so, of vehicles from B to make room for the storage of movement C to G. During phase 2, this is accomplished by a lagging green at intersection 1 beyond that at intersection 2. For an assumed speed of about 20 m.p.h. (30 feet per second) and a distance of 200 feet, the lag is 7 seconds. Thus the green interval for approach B is $35-7=28$ seconds, or $G/C=0.47$.

Through and right-turn movement: From chart 2, design capacity=1,100 v.p.h. This is satisfactory since the load, 1,100+50= 1,150 v.p.h., is only slightly above design capacity.

Left-turn movement: From chart 8B, design capacity of left-turn lane is about 95 v.p.h.; left-turn volume is 30 v.p.h. Required length of left-turn lane for this volume, from chart 8C, is 65 feet.

Approach A—phase 2

From chart 2, for a width of 36 feet and $G/C=0.47$, (same as at B), design capacity=1,070 v.p.h.; traffic load=900 v.p.h.

Approach C—phase 1

Available green time per cycle is $60-28-6=26$ seconds, or $G/C=0.43$. It is first necessary to check the capacity of left turn in accordance with Special Conditions, item

5. From chart 8A, using V_0=120 v.p.h., T_0=15%, G/C=0.43, and T_3=15%, capacity of left turn is found to be 280 v.p.h., compared to a volume of 180 v.p.h. The capacity of the approach, as a whole, is obtained from chart 2. Since $L+(R/2)$ is greater than 35, R=10% and L=30% are used in the solution (see single-asterisk note in chart 2); and for the 20-foot approach, design capacity=370 v.p.h.; traffic load=330 v.p.h.

Storage space occupied by movement C to G on approach G during phase 1 may be obtained from chart 10B. For a volume of 180 v.p.h., the length required for storage, if in a single lane, is 135 feet.[6] In three lanes the length occupied would be about 45 feet. Available length on approach G is approximately 100 feet.

Approach F—phase 1

As above, G/C=0.43. The left-turn volume obviously can be handled. From chart 2, for an approach width of 22 feet, design capacity = 480 v.p.h. Since this is much in excess of the traffic load, the green interval on this approach could be decreased to give, in effect, a short advance green to movement C to G.

Length of storage on the two lanes of approach F for a volume of 160 v.p.h., from chart 10B, is 115÷2, or about 60 feet.[6] Available length is approximately 120 feet. This leaves sufficient space ahead of the stop line at D to consider the exit from approach D unimpeded in regard to capacity.

Approach D—phase 2

Movement D to K flows freely at all times. The green interval for left-turn movement D to F is 60−25−6=29 seconds (25 seconds is the green interval on approach J, previously determined), and G'/C=0.48. From chart 10A, design capacity of movement D to F is about 350 v.p.h. Since this is much in excess of the volume, the lagging green on approach J, phase 1, could be increased. Required length of left-turn lane, from chart 10B, is 120 feet.

Exit E—phase 1

Since exit E receives traffic from J and C simultaneously, the capacity of this combined movement should be checked as controlled by the exit (see Special Conditions, item 7). Total volume during phase 1 is 260+110=370 v.p.h. From chart 2, using $W/2$=22, no turns, etc., and G/C=0.42, design capacity of exit=620 v.p.h.

According to the above analysis, the intersection design is found to be satisfactory for operation during the evening peak hour. A similar capacity check should be made for the morning peak hour, which may show a different signal timing or a need for some modification in the geometric layout. Length of turning lanes and other storage areas would be based on the larger of the two values determined for the morning and evening peaks.

10. Interpolation in Charts

Where the intersection is in an area having characteristics between those of a downtown and an intermediate area, interpolation is made between the capacity values of charts 2 and 3, or charts 4 and 6.

Where the characteristics of the intersecting facility are between those of a street and an expressway, capacity values can be interpolated between those of charts for streets and those for expressways.

11. Use of City or Local Factor

In localities where driver characteristics, or other conditions, are believed to be different from those represented by the basic data in the Manual, a further adjustment to the Manual or chart values may be applicable. This adjustment can be expressed as a "city factor," established by relating actual hourly volumes measured at existing intersections loaded to their possible capacity (continual backlog of waiting vehicles on the approach during one hour), to the possible capacity, for comparable conditions, obtained from the Manual (1.10 times the value given in figure 24 therein and adjusted as necessary for specific conditions). Such field measurements should include a sufficient number and type of intersections to be representative for the city or locality as a whole. The ratio of measured possible capacities to those calculated by the Manual methods gives the "city factor." The numerical value of this factor may be a constant for all intersection approach conditions, or it may vary with the type of area and parking regulation.

Part II—One-Way Streets

Separate design capacity charts for one-way streets could have been developed but because of the definite relation between the capacity of two-way and one-way streets, this was considered unnecessary. Instead, a procedure is given for evaluation of intersection capacities of one-way streets by use of the charts for two-way streets.

DESIGN CAPACITY FACTORS

The basic data for intersection capacities of one-way streets, expressed in terms of average maximum volumes, are shown in figure 10.[7] This chart gives the same type of information for one-way streets as figure 1 for two-way streets. Since the same average conditions are represented in both, a definite relation can be established for the four upper curves of figure 10 and the comparable curves of figure 1. Thus, a series of factors to convert the maximum volumes accommodated by one approach on two-way streets to the maximum volumes on one-way streets can be obtained by making a ratio, for comparable conditions, of the volumes in figure 10 to the volumes in figure 1. For example, in figure 10, for an approach (one-way street) width of 34 feet, in an intermediate area, with no parking, a volume of 2,500 vehicles per hour of green is given; in figure 1 for the same approach width (68-foot two-way street) under the same conditions, a volume of 2,000 vehicles per hour of green is given. The conversion factor thus is 2,500/2,000=1.25.

Table 3 shows these conversion factors, i, for the range of approach widths up to 50 feet. It is to be emphasized that the parking on a one-way street represents parking on one side only. This condition is comparable to one approach of a two-way street, and so permits use of the two-way capacity charts. To find the design capacity of an approach on the one-way street, use charts 1–6, with $W/2$ equal to the whole width of the one-way street; then multiply the given K by appropriate i in table 3. Design capacities and required lengths of separate turning lanes on one-way streets are the same as those on two-way streets, for which charts 7–10 are used.

When parking exists on both sides of a one-way street the operating conditions are not similar and the two-way values and adjustments cannot be used. In such cases reference should be made to item IV on page 90 of the Manual for adjustments to the values in figure 10. It will be noted that the values in figure 10, for a given width of street, show one-way street capacity with parking on both sides to be about 70–75 percent of that with parking on one side in downtown areas, and about 80–85 percent in intermediate areas.

RELATION OF DESIGN CAPACITY TO POSSIBLE CAPACITY

The relation between design and possible capacities developed for two-way streets also is applicable to one-way streets. The factors f from table 2 apply directly when used with $W/2$ as the whole width of a one-way street. Factors for the condition "with parking" are applicable to parking on one side only of one-way streets. The procedure to obtain possible capacity of a one-way street is: (1) determine design capacity for an equivalent approach from charts 1–6; (2) multiply by factor i in table 3; and

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[6] Storage space from chart 10B is based on 1.5 times the average number of vehicles storing per cycle. Actually the maximum that may be stored can be as high as two times the average number per cycle. Thus, where feasible, the length of such storage area should be predicated on chart 12E. In this problem, however, the space is adequate on either basis.

[7] Figure 26, p. 84 of the Manual.

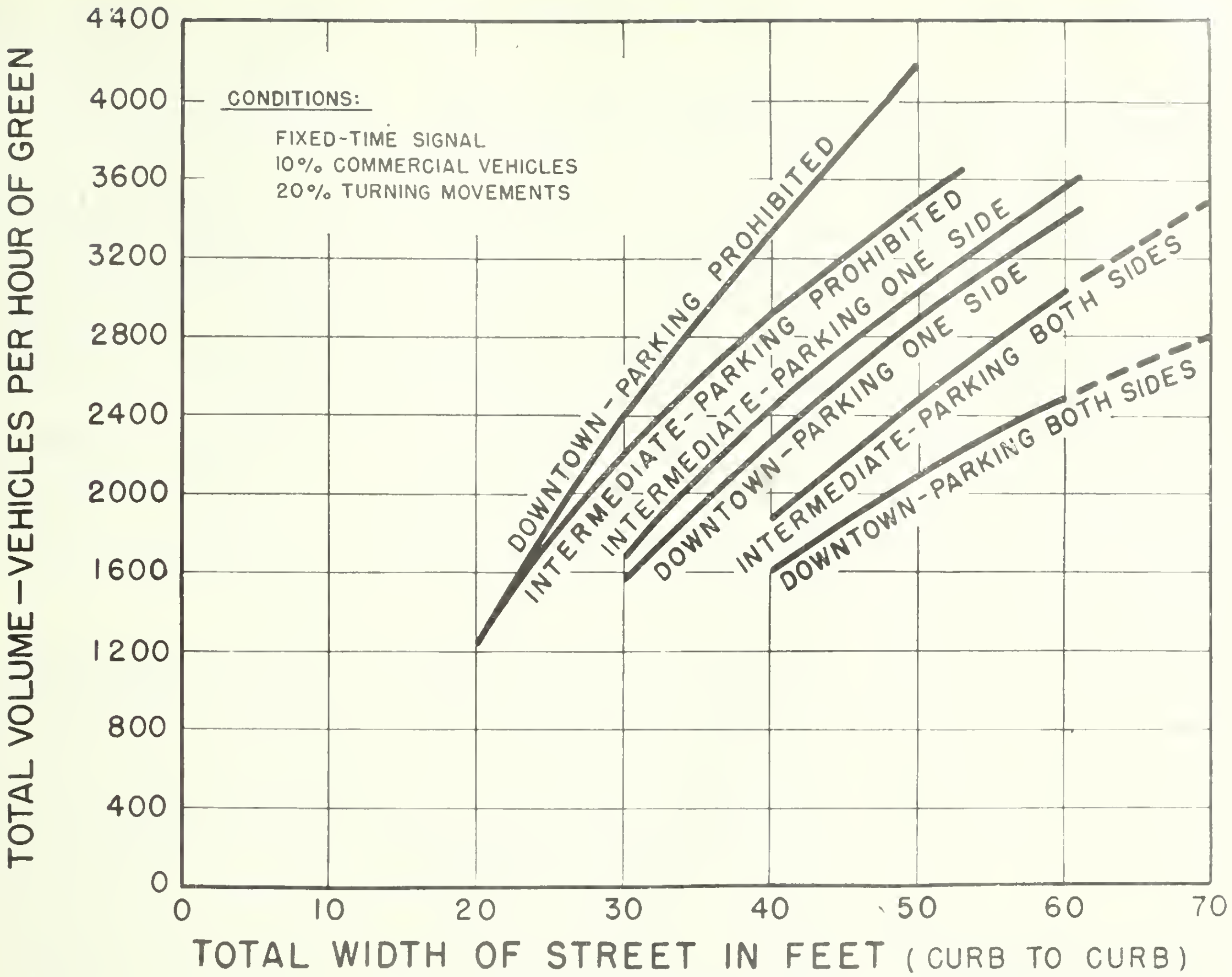

Figure 10.—Average maximum volumes at intersections on one-way streets, for different widths and by type of area and parking regulation.

(3) multiply by factor f in table 2. Possible capacities of separate turning lanes on one-way streets are the same as those on two-way streets; i. e., $1.2K_2$ or $1.2K_3$.

PROCEDURE

The following procedures, employing charts 1–10 and tables 2 and 3, are used to determine the capacity of one-way streets. For ready reference, each major condition is treated separately.

A.—Average Conditions

Proceed through chart 1 in the normal manner, with $W/2$ equal to the width of one-way street, and obtain a capacity value. Multiply this by factor i, table 3, to obtain design capacity of one-way street. If the chart is entered on the right with a given volume, in order to obtain either a value of G/C or $W/2$, divide this volume by factor i before entering the chart.

Problem 27

An intersection on a one-way street, 30 feet wide between curbs, with parking on one side, located in a downtown area, is assumed to be operating under average conditions. What is the design capacity if the signal is so timed that $G/C=0.60$? What will be the possible capacity?

Solution: In chart 1, using $W/2=30$, curve III, and $G/C=0.60$, read = 600 v.p.h. From tables 3 and 2, $i=1.25$ and $f=1.40$.

$K=600\times1.25=750$ v.p.h.

$P=750\times1.40=1{,}050$ v.p.h.

Problem 28

If, in problem 27, the approach volume to be accommodated is only 625 v.p.h., to what may the ratio of G/C be reduced?

Solution: Enter chart 1 with $W/2=30$, proceed to curve III, and project a line downward. Then enter the chart with a volume of $625\div1.25=500$ v.p.h., proceed to the left and intersect the vertical line previously projected; read $G/C=0.50$.

Table 3.—*Factor i to convert capacity value (charts 1–6) on one approach on a two-way street to that on a one-way street*[1]

$W/2$ = Approach width [2]	No parking		Parking on one side	
	Downtown	Intermediate	Downtown	Intermediate
Feet				
20	0.88	0.94	...	...
22	.94	1.00	...	...
24	.98	1.04	...	...
26	1.02	1.09	1.19	1.09
28	1.05	1.13	1.22	1.13
30	1.08	1.18	1.25	1.17
32	1.11	1.22	1.29	1.21
34	1.13	1.25	1.31	1.24
36	1.15	1.29	1.34	1.26
38	1.17	1.32	1.37	1.28
40	1.19	1.35	1.40	1.30
42	1.21	1.38	1.42	1.32
44	1.23	1.41	1.44	1.33
46	1.25	1.44	1.46	1.35
48	1.26	1.47	1.47	1.36
50	1.27	1.48	1.48	1.38

[1] For the same width of approach, type of area, and parking regulation.

[2] For one-way streets, the total curb-to-curb width, exclusive of separate turning lanes; for two-way streets, one-half (normally) of the curb-to-curb width.

B.—No Parking, in Downtown or Intermediate Area

Enter chart 2 or 3 with $W/2$ equal to the width of the one-way street. Proceed through the chart in normal manner, but instead of actual L use $(L/2)+5$. Multiply the result by factor i from table 3 to obtain design capacity of the one-way street. The single-asterisk note in charts 2 and 3 does not apply to one-way streets.

Problem 29

A 40-foot street in a downtown area is converted to one-way operation with no parking. Other conditions are $T=18\%$, $R=12\%$, $L=20\%$, $B=$no bus stop, and 35% of the cycle time must be devoted to the cross street. What is the design capacity of the one-way street if two-phase signal control is used with $C=60$ seconds and each amber$=3$ seconds? What will be the possible capacity?

Solution: Green time that must be allotted to the cross street is 35% of 60=21 seconds. Green time available for the one-way street is $60-21-(2\times3)=33$ seconds, and $G/C=33/60=0.55$.

Using chart 2, with $W/2=40$, $T=18\%$, $R=12\%$, $L=(20/2)+5=15\%$, $B=$no bus stop, and $G/C=0.55$, obtain a capacity value of 1,100 v.p.h. From tables 3 and 2, $i=1.19$ and $f=1.40$.

$K=1{,}100\times1.19=1{,}300$ v.p.h.

$P=1{,}300\times1.40=1{,}820$ v.p.h.

C.—With Parking on One Side, in Downtown or Intermediate Area

Enter chart 4 or 6 with $W/2$ equal to the width of the one-way street. Proceed through the chart in normal manner, except instead of actual L use $(L/2)+5$. Use supplemental chart 5 in normal manner, as for two-way streets. Multiply the result from chart 4 or 6 by factor i from table 3 to obtain capacity of the one-way street. The single-asterisk note on charts 4 and 6 does not apply.

Problem 30

If, in problem 29, all of the conditions remain the same except that parking is permitted on one side and $D=20$ feet, what will be the design capacity?

Solution: First use chart 5C, with $R+L=12+20=32\%$, and $D=20$, obtaining $Z=-6$. Then, from chart 4, a capacity value of 640 v.p.h. is found. In table 3, $i=1.40$ for a 40-foot approach with parking on one side.

$K=640\times1.40=900$ v.p.h.

D.—With Separate Turning Lanes, No Separate Signal Indication

With right-turn lane: Follow the instructions given on chart 7, except in obtaining the capacity of the combined through and left-turn movement M_2 use $(L/2)+5$ instead of actual L in chart 2 or 3; multiply this by factor i from table 3.

With left-turn lane: Use chart 7, since chart 8 is not applicable to one-way streets. In obtaining the capacity of the combined through and right-turn movement, use $L=5\%$ (instead of 0%) in chart 2, 3, 4, or 6; multiply this by factor i from table 3.

With both right- and left-turn lanes: Follow the instructions on chart 9, except in obtaining the capacity of the through movement M_1 use $L=5\%$ (instead of 0%) in the solution on charts 2 and 3; multiply this by factor i from table 3.

Problem 31

If, in problem 29, all of the conditions remain the same, except that the approach is widened by addition of a left-turn lane with an adequate curb return (and there is little if any pedestrian interference), what will be the design capacity?

Solution: The design capacity of the left-turn lane, from chart 7B, using $G/C=0.55$ and $T_3=18\%$ (assuming the percentage of trucks is the same for all movements), is 360 v.p.h.

A capacity value for the combined through and right-turn movement, from chart 2, using $W/2=40$, $T=18\%$, $R=12\%$, $L=5\%$, $B=$no bus stop, and $G/C=0.55$, is 1,220 v.p.h. From table 3, $i=1.19$. The design capacity of the through plus right-turn lanes$=1{,}220\times1.19=1{,}450$ v.p.h.

From step 3, chart 7, left-turn volume$=(1{,}450\times20)\div(100-20)=360$ v.p.h., which is the same as the design capacity found above. Design capacity of approach $=1{,}450+360=1{,}810$ v.p.h. The length of left-turn lane required, from chart 7C, is approximately 200 feet.

E.—With Separate Turning Lanes and Separate Signal Indication

With right-turn lane: Follow the instructions on chart 10, except for step 2 in the series on the right side of the figure substitute the following: Obtain design capacity of the combined through and left-turn movement M_2 in the usual manner from charts 2–6. Use $W/2$ as the normal width of approach, exclusive of turning lane; $(L/2)+5$ instead of actual L; and $R=0\%$. Multiply the result obtained by factor i from table 3.

With left-turn lane: Follow the instructions on chart 10, except for step 2 in the series on the left side of the figure substitute the following: Obtain design capacity of the combined through and right-turn movement M_3 in the usual manner from charts 2–6. Use $W/2$ as the normal width of approach, exclusive of turning lane, and $L=5\%$. Multiply the result by factor i from table 3.

F.—Special Conditions

The items listed under the heading Special Conditions for two-way streets (page 113) also apply to one-way streets, except that the charts are to be used as described above in sections B–E.

Problem 32

What is the design capacity, in problem 29, if all of the conditions remain the same except that a bus stop is provided at the intersection on the one-way street with approximately 90 busses stopping per hour, and T, exclusive of stopping busses, is 5%? What is the possible capacity?

Solution: According to item 1, page 113, $W/2=40-12=28$ feet. Using this in chart 2, with $T=5\%$, $R=12\%$, $L=(20/2)+5=15\%$, bus-stop condition line B_x, and $G/C=0.55$, a capacity value of 890 is obtained. From tables 3 and 2, $i=1.19$ and $f=1.40$.

Design capacity$=(890\times1.19)+90$ busses $=1{,}150$ v.p.h.

Possible capacity$=(1{,}060\times1.40)+90$ busses$=1{,}570$ v.p.h.

Problem 33

What is the design capacity in problem 25, page 114, if all of the conditions remain the same, except that the 40-foot intercepted street is converted to a one-way street for travel in the westerly direction?

Solution: According to item 8, page 114, the left-turn movement is considered to be equivalent to the through movement, so that in the solutions $L=(0/2)+5=5\%$ (see section B above). Using chart 2 with $W/2=40$, $T=10\%$, $R=30\%$ or more, $L=5\%$, $B=$no bus stop, and $G/C=0.40$, a capacity value of 880 is obtained. From table 3, $i=1.19$. Design capacity of the one-way east approach$=880\times1.19=1{,}050$ v.p.h.

Part III—Expressways

FEATURES OF EXPRESSWAYS

An expressway is defined as a divided arterial highway for through traffic with full or partial control of access and generally with grade separations at intersections.[8] The salient geometric features of an expressway are: a divided highway designed to high standards, insulated for the most part from the adjacent development; shoulder space for emergency use (no parking adjacent to the traveled way); bus stops (if any) on separate turnouts; and properly designed and controlled intersections. Where partial control of access is used, the expressway will intersect some streets or highways at grade. These intersections will require added turning lanes of adequate design, pedestrian cross-walk

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[8] Definition adopted by the American Association of State Highway Officials, June 25, 1949.

Table 4.—*Minimum lengths of speed-change lanes and taper to or from a stop position*

Highway design speed	For deceleration lanes			For acceleration lanes [1]		
	Length exclusive of taper	Length of taper	Total length	Length exclusive of taper	Length of taper	Total length
Miles per hour	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
30	15	100	115	....	140	140
40	40	125	165	75	175	250
50	90	150	240	280	200	480
60	130	175	305	550	250	800

[1] Applies only to conditions with widened approaches or to intersections without signal control.

controls, and in some instances traffic signal controls for all traffic.

On expressways, where the above conditions are satisfied, intersection capacities expressed in terms of vehicles per hour of green per unit of width will be higher than on ordinary streets or highways, and the capacity data represented on charts 1–10 are not applicable. Charts 11, 12, and 13 are design capacity adjustments for specific conditions on high-type facilities, based on the data in item V on page 90 of the Manual. These charts are applicable to both rural and urban conditions, and to divided two-way highways or to one-way facilities. Two general conditions govern and are treated separately: charts 11 and 12 are applicable where separate turning lanes exist; and chart 13 where widened approaches are used.

EXPRESSWAYS WITH SEPARATE TURNING LANES

Charts 11 and 12 give all of the necessary information for evaluating the expressway design capacity at signalized intersections having separate turning lanes (the type of layout shown in the upper right-hand corner of chart 12). Added turning lanes are arranged for the exclusive use of turning vehicles, and other traffic cannot use them to proceed through the intersection. Added lanes designed to permit their use by through traffic are discussed in the next section, concerning widened intersections.

Chart 11 gives the solution for the design capacity of the through movement K_1 on the expressway at a signalized intersection. It is based on the Manual value of 1,000 passenger vehicles per hour of green per 10 feet of lane width. With adjustment for the percentage of trucks and busses and for the G/C value, the design capacity can be read directly for the width of through lanes ($W/2$ on the sketch in chart 12). This capacity value must be used jointly with separately determined capacities for right- and left-turn movements obtained from chart 12. The procedure is the same as that described for charts 9 and 10.

Chart 12 gives solutions for design capacities of separate turning lanes, in which charts A, B, and C are for controls without a separate signal indication and chart D with a separate signal indication. Since the conditions are identical with those in chart 8, the design capacity of a left-turn lane is obtained from chart 12A or 12B, the larger value governing. Chart 12C is of the same form as that of chart 7B, but is based on a control value of 1,000 vehicles per hour of green per 10 feet of width instead of 800. A third adjustment is included in chart 12C in terms of three degrees of pedestrian interference. Chart 12D is the same as chart 10A except that the control value of 1,000 is used instead of 800.

The length of right- or left-turn lane, with or without separate signal indication, is given in chart 12E and predicated on the maximum number of vehicles that can be stored per cycle, which is assumed to be twice the average number. Since expressways are intended for rapid vehicular movement with a minimum of operational delay, the length of turning lanes should be predicated on the likely approach speeds of traffic during the green signal periods. For this purpose the turning lanes should be sufficiently long to permit turning vehicles to decelerate to the safe speed of the turn, with allowance for drivers to bring their vehicles to a stop if necessary. Minimum lengths of deceleration lanes to allow for such operation are given in table 4. Comparative values are obtained from chart 12E and from the second column of table 4 and the larger of the two should be used in design. The length of taper should never be less than that shown in the third column of table 4.

On expressways, bus stops in the vicinity of cross streets should be located off the through lanes and on the far side of the intersection. When thus positioned at an intersection with corner islands, as in the sketch on chart 12, bus stops have little if any effect on intersection capacity. Thus, charts 11 and 12 apply to intersections with bus stops on the far side as well as to those with no bus stops. In the event that a near-side bus stop is provided, the capacity should be reduced according to the adjustment in item V-3B on page 91 of the Manual.

Another condition adjustment is shown in item V-2A (2) in the Manual, for right-turn movements as affected by frontage-road traffic. When a frontage road is sufficiently close to affect a right-turn movement, the capacity condition is the same as that of a left turn from an added lane without a separate signal indication, and values from charts 12A and 12B are applicable.

Intersections on expressways should be designed so that anticipated volumes will not exceed the design capacity. For certain combinations of traffic volumes and distributions, however, it may not be practicable to accommodate each approach movement at design capacity. One of the turning movements may have to operate above design capacity, but the excess should not be large. The relative amount can be determined by calculating possible capacity.

On expressways, possible capacity for any movement may be obtained by multiplying the design capacity by 1.2.

Problems 34–37 illustrate the use of charts 11 and 12.

Problem 34

On the expressway shown in figure 11, traffic from east to west during the peak hour is approximately 60% of the total two-way traffic on the expressway. On the east approach, what is the design capacity of the through lanes, the right-turn lane, and the left-turn lane, when trucks comprise 10% of each movement, $C=65$ seconds, and $G=39$ seconds?

Solution: Design capacity of the through pavement, K_1, using chart 11 with $W/2=25$, $T=10\%$, and $G/C=39/65=0.60$, is 1,350 v.p.h.

Design capacity of the right-turn lane K_2, using chart 12C with $G/C=0.60$, $a=11$, $T_2=10\%$, and light to moderate pedestrian interference, is 530 v.p.h.

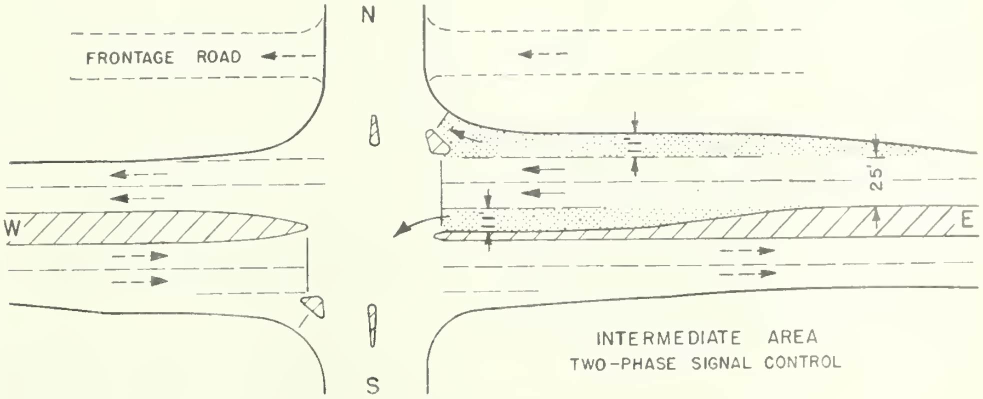


Figure 11.—Illustrative problems 34–37.

Design capacity of the left-turn lane K_3 is determined by the larger of the two values from charts 12A and 12B. The volume opposing the left turn V_0, as used in chart 12A, is 40% of the total peak-hour volume on the expressway, or $1{,}350 \times 40/60 = 900$ v.p.h. Using this in chart 12A gives a zero capacity. Minimum capacity in chart 12B, with $C=65$ seconds, is 88. Thus, the value in chart 12B governs and $K_3=90$ v.p.h. (using a rounded figure).

Problem 35

Determine the minimum signal timing and lengths of turning lanes of the east approach of the intersection shown in figure 11 if the design speed is 50 m.p.h. and the design volumes during the evening peak hour are as follows: through movement, 1,120 v.p.h. of which 5% are trucks; right-turn movement, 280 v.p.h. of which 30% are trucks; and left-turn movement, 90 v.p.h. of which 2% are trucks.

Solution: The proportion of green time, G/C, needed for the through movement is obtained from chart 11: Using $W/2=25$, $T=5\%$, and an approach through volume of 1,120 v.p.h., the required $G/C=0.47$.

For the right-turn movement chart 12C is used. Entering the chart at the bottom with a volume of 280 v.p.h., and using light to moderate pedestrian interference, $T_2=30\%$, and $a=11$, the required $G/C=0.38$.

It is apparent from the approach volumes indicated and from the distribution of traffic by direction, as shown in problem 34, that the capacity of the left-turn lane is not governed by the condition in chart 12A. Thus, using chart 12B, it is found that for a left-turn volume of 90 v.p.h. a cycle length C of 64 seconds is required.

The through movement, for which the required $G/C=0.47$, governs the design capacity of the east approach. Assuming that a 64-second cycle is satisfactory, $G=64 \times 0.47=30$ seconds and, if each amber period is 3 seconds, the signal timing will be 30 seconds green and 6 seconds amber, leaving 28 seconds red (green on the cross street).

In the event that a longer cycle had to be used, say 75 seconds, the design capacity of the left-turn lane (chart 12B) would be 77 v.p.h. The 90 v.p.h. could still be handled, although not as satisfactorily, since possible capacity would be $77 \times 1.2=92$ v.p.h.

The lengths of turning lanes, from chart 12E, required to handle the volumes indicated are:

Right-turn lane: Using $V_2=280$ v.p.h., $C=64$ seconds, and $T_2=30\%$; $D_2=300$ feet.

Left-turn lane: Using $V_3=90$ v.p.h., $C=64$ seconds, and $T_3=2\%$; $D_3=80$ feet.

The length of each turning lane required for speed change, from table 4, is 90 feet plus a taper of 150 feet. Thus, the right-turn lane should be 300 feet long plus a 150-foot taper, and the left-turn lane 90 feet long plus a 150-foot taper.

Problem 36

If, in problem 35, all of the conditions remain the same, except that a one-way frontage road is provided (dash lines in figure 11), what will be the design capacity of the right-turn lane on the east approach when a volume of 250 v.p.h. on the frontage road, of which 12% are trucks, receives a green signal at the same time as the expressway traffic?

Solution: In this case it is necessary to check the capacity of the right-turn lane as affected by the crossing of frontage-road traffic, which can be determined from charts 12 A and 12 B (see page 119, cols. 2-3).

Using $V_0=250$ v.p.h. in chart 12A as the volume conflicting with the right-turning movement, $T_0=12\%$, $G/C=0.47$, and T_3 (which in this case is the percentage of trucks in the right-turn movement) $=30\%$, the design capacity, as controlled by the frontage road traffic, is found to be 190 v.p.h.

This indicates that, with a frontage road, a right-turn volume of 280 v.p.h. cannot be accommodated, and three-phase control is needed at the intersection of the frontage road and the cross street.

Problem 37

If, in problem 35, all of the conditions remain the same, except that the left-turn volume on the east approach is 140 v.p.h., what will be the signal timing if the left turn is to be accommodated on a separate signal indication? The 28-second green interval on the cross street is to be retained to give pedestrians sufficient time to cross the street. Left-turn movement on the west approach also will move on this phase.

Solution: In problem 35 it is shown that a value of $G/C=0.47$ is required for the through movement, during which the right turn will also be accommodated. The separate phase for the left turn will require an additional $G'/C=0.13$, as determined from chart 12D with a volume of 140 v.p.h., $T_3=2\%$, and $a=11$ feet.

To retain the 28-second green period on the cross street, the length of cycle must be: $C=0.47C+0.13C+28+(3 \times 3)$ amber$=$ 93 seconds.

The green signal times are thus: For through and right-turn movement on the expressway, $93 \times 0.47=44$ seconds; for left turns on the expressway, $93 \times 0.13=12$ seconds; for the cross street, 28 seconds. Each of the three phases is followed by a 3-second amber period.

It may be noted that, while the 28-second green interval has been retained for the cross street, the proportion of green time (G/C) for movement of traffic on this street, because of lengthened cycle, has been decreased; thus, the capacity of the cross street is less than that in problem 35.

EXPRESSWAYS WIDENED THROUGH INTERSECTIONS

On expressways where the cross roads at grade are widely spaced it may be necessary to widen the expressway pavements at the intersections to prevent their capacity from being diminished at these points. Such treatment is illustrated by a sketch in the upper right-hand corner of chart 13, and from this chart the design capacity can be determined or the number of lanes required to handle a given volume can be established.

In this condition the added lanes, shown as e_2 and e_2 in the sketch, are carried through the intersection, unobstructed by triangular islands. When the expressway is operating at relatively low volumes, through traffic will proceed through the intersection on the normal width of pavement, $W/2$, exclusive of widening. During such times the added lanes e_2 and e_3 will function as speed-change lanes for turning vehicles. At or near design capacity, however, through traffic will be stored on the added lanes and proceed through the intersection when the signal changes. Turning vehicles will also use these lanes at the same time. For this reason an adjustment in capacity must be made. This is accounted for in chart 13.

An adjustment for bus stops is not included in chart 13. Since this type of intersection generally is used to maintain the high capacity available on other portions of the expressway without intersections at grade, bus stops should be excluded from all lanes intended for regular traffic use. If bus stops are necessary at such locations, they should be provided on frontage roads or on separate turnouts.

Chart 13 is constructed on the same basis as chart 11 (see pp. 87 and 103 of the Manual), with a design capacity of 1,000 vehicles per hour of green per 10 feet of width. The upper left and lower right portions of chart 13 are identical with chart 11. The added adjustments for turning movements are the same as in previous charts, the percentage reduction being $0.5R$ for right turns and $1.0L$ for left turns.

For this capacity condition the widened approach pavement must be of sufficient length to encourage its use by through traffic, as well as be adequate for deceleration and storage of turning vehicles. Controls for the required lengths are given in the notes below the sketch in the upper right corner of chart 13. The widened length in feet in advance of the intersection, shown as D_a, should be at least five times the green interval in seconds (see Manual, p. 89) and also should be adequate for deceleration purposes, as indicated in the second column of table 4. In addition it should have a proper taper, as shown in table 4.

For the maneuvering of traffic, the widened pavement D_b beyond the intersection should be of length somewhat longer than on the approach side, for which a factor of 1.5 is assumed. Further, the length should be checked for suitability for acceleration, according to the fifth column of table 4. The sixth column of table 4 gives the needed length of taper.

Problem 38

An expressway with two one-way pavements, separated by a median about 150 feet wide, intersects a cross road at grade, and a signal must be installed. If the normal width of each expressway pavement is 33 feet, and at the intersection each pavement is widened by 12 feet on the left and on the right for an adequate distance in advance of and beyond the intersection, what is the design capacity of one approach, if $T=10\%$, $R=15\%$, $L=12\%$, and $G/C=0.54$?

Solution: The total width of approach is $W/2+e_2+e_3=33+12+12=57$ feet. Using this in chart 13 with the conditions given above, $K=2{,}260$ v.p.h. The wide median, in effect, separates the expressway into two one-way facilities; therefore, it is not necessary to check for the capacity of the left turn.

Problem 39

An urban expressway in rolling terrain, designed for a speed of 50 m.p.h., has two 12-foot lanes in each direction, and all cross streets are separated in grade except at one isolated intersection. The volume of traffic on the cross street can be accommodated by a two-phase traffic signal with 30% of the elapsed time allowed for the cross street, and a 20-second green interval for pedestrians.

If the intersection treatment is to be of the type shown in the upper right-hand corner of chart 13, determine the minimum number of lanes on the expressway, at the intersection, that will enable the intersection approaches to accommodate a volume of traffic equal to the capacity of the two 12-foot lanes where flow is uninterrupted. On the critical approach, during the peak hour, $T=15\%$, $R=18\%$, and $L=4\%$.

Solution: On the basis that 30% of the cycle is required for the cross movement with a green interval of 20 seconds, the shortest cycle that can be used is $20\div0.30=67$ seconds. Allowing 7 seconds for the amber periods, the green interval available for expressway traffic is $67-7-20=40$ seconds, and $G/C=40/67=0.60$.

The capacity of the expressway on the portions where the flow is uninterrupted, with 15% trucks in rolling terrain, is found to be 1,050 v.p.h. per lane,[9] or a one-way flow on the facility of 2,100 v.p.h. The total number of lanes on the one approach to handle this volume can be obtained from chart 13. Entering the chart at the bottom with a volume of 2,100 v.p.h. and using $G/C=0.60$, $L=4\%$, $R=18\%$, and $T=15\%$, the total approach width, $W/2+e_2+e_3$, is found to be 47 feet.

Four lanes will therefore be required on the one approach at the intersection, or an extra lane on each side of the normal pavement, to accommodate a volume equal to the uninterrupted capacity flow of the expressway.

Since the expressway is a two-way facility it is necessary to check separately in chart 12 for capacity of the left-turn lane. Obviously chart 12B governs, from which it is found for $C=67$ seconds that 86 left-turning vehicles can be handled. This is satisfactory since 4% of 2,100 is 84 v.p.h.

The minimum lengths of widened pavement, from the notes under the sketch on chart 13, should be: For D_a, 200 feet plus 150-foot taper; for D_b, 300 feet plus 200-foot taper.

---

[9] 1,500 X 0.70 = 1,050 v.p.h. Practical or design capacity of urban expressways is given as 1,500 vehicles per hour per lane on p. 47 of the Manual; 0.70 is a factor interpolated in table 9, p. 56 of the Manual, for the effect of commercial vehicles on practical capacity.

Part IV—Over-All Intersection Capacity

USE IN PRELIMINARY DESIGN

In planning or in preliminary design there often is need for a quick, approximate determination of intersection capacities. The problem usually resolves itself into one of two conditions: (1) where the intersecting volumes are known and street widths are established, to determine whether capacity is adequate; or (2) where the intersecting volumes and the width of one street are given, to determine the width of the intersecting street.

These problems can be solved with charts 1–13 by first determining the proportion of green time required for one approach, assuming a cycle length and appropriate amber periods; then calculating the resultant G/C for the cross-street approach; and finally determining, according to the cross-street volume, either its adequacy for capacity or its necessary width.

SOLUTION WITH HOURLY TRAFFIC VOLUMES

Chart 14 was devised for use in planning of systems or for early stages of design, as well as for review of preliminary plans, where a quick method is needed for determining intersection capacities or required street widths at a four-way intersection under signal control. Chart 14 combines the necessary information for both of the intersecting streets on one chart and gives results in terms of over-all capacity. It takes into account jointly, for average conditions, the intersection of any two facilities, regardless of the type of each (one-way or two-way street or expressway), type of area, and parking regulation. The left half of the chart is used for the approach on one street and the right half for the approach on the other (intersecting) street. A line projected between the inner sides of the two charts determines, at the point where it intersects the center axis, y-y, the adequacy of intersection capacity.

The two parts of chart 14 are identical except for the reverse plotting. The arrangement of each part is similar to that of chart 1, but the G/C ratio is made the outer scale and the volume is shown as the lower series of curves. In addition to the four area–parking conditions shown on chart 1, conditions for one-way streets and expressways are represented in the upper parts of chart 14 by other sloping lines. These are control values previously presented, including adjustments to obtain design capacity. Notes at the lower left show the proper $W/2$ value to be used for each case. The G/C ratio on the inner side scales is the proportion of time required on the one approach for operation at design capacity. With an assumption of 10 percent of the cycle time being used in the amber periods, design capacity is obtained when the total of two green intervals is 90 percent of the cycle (the sum of the two G/C values$=0.90$). The zero point on the y-y axis is located so that a straight line between any two G/C values passes through it when their sum is 0.90. The scale values above and below the zero point on the y-y axis show the proportion by which the sum of the G/C values is deficient or in excess of the design capacity condition. As indicated on the chart, a design capacity deficiency of 20 percent is about the possible capacity condition.

With chart 14, graphic solutions can be made for various combinations of control conditions to determine one missing factor. It must be remembered that this solution is for average conditions only, with assumptions that trucks and busses constitute 10 percent and turning movements on streets 20 percent (15 percent on expressways) of the total approach volume. Where specific conditions are otherwise, the solutions should be obtained from charts 2–13.

Problem 40

In a downtown area a two-way street (approach A) 62 feet wide with parking intersects a two-way street (approach B) 44 feet wide without parking. A fixed-time signal is used and conditions are assumed to be average. When the peak-hour volume on approach A is 400 v.p.h. in one direction and on approach B is 600 v.p.h. in one direction, is the capacity of the intersection adequate?

Solution: For approach A, enter chart 14 at the left with $W/2=62/2=31$, proceed to the right to the curve for downtown two-way street with parking, then down to an approach volume of 400 v.p.h., and to the right to $G/C=0.38$. For approach B, enter the chart at the extreme right with $W/2=$

44/2=22, proceed to the left to the curve for downtown two-way street without parking, then down to an approach volume of 600 v.p.h., and to the left to $G/C=0.47$. A straight line between the two values of G/C falls below the zero point on the y-y axis so capacity is adequate; in fact, there is about 6% excess capacity.

Adjusted signal timing, if desired, can be obtained by dividing each G/C by the portion of capacity required in respect to design capacity. In this case, with 6% excess capacity, the factor is $(100-6) \div 100=0.94$, and the signal timing would be: For approach A, $0.38 \div 0.94=0.40$; for approach B, $0.47 \div 0.94=0.50$; for amber (the remainder), 0.10.

Problem 41

If, in problem 40, all of the conditions remain the same except that approach B has to accommodate a peak-hour volume of 930 v.p.h. in one direction, to what extent must this approach be widened to make the intersection operate at design capacity?

Solution: For approach A, use the left portion of the chart as in the previous example, obtaining a value of $G/C=0.38$. From this point project a straight line through the zero point (design capacity) of line y-y to intersect the G/C scale on the right portion of the chart. From this point ($G/C=0.52$), proceed to the right to an approach volume of 930 v.p.h., then up to the curve for downtown two-way street without parking, and to the right to $W/2=30$. Approach B would have to be widened to 60 feet if the intersection is to operate at design capacity.

Problem 42

A two-way expressway (approach A) with two 24-foot pavements and added turning lanes intersects a 52-foot one-way street without parking (approach B). The intersection is situated in an intermediate area and is to be controlled by a fixed-time signal. The plan is in a preliminary design stage with only general traffic information of 1,400 v.p.h in one direction on approach A and 1,200 v.p.h. on approach B. Can these volumes be handled satisfactorily?

Solution: Enter chart 14 at left with width of approach A=24 feet, proceed to the right to the curve for expressway with turning lanes, then down to an approach volume of 1,400 v.p.h., and to the right to $G/C=0.56$. Enter the chart at right with a width of approach B=52 feet, proceed to the left to one-way street without parking in intermediate area, then down to an approach volume of 1,200 v.p.h., and to the left to $G/C=0.37$. A straight line between the two values of G/C shows that the intersection will operate at slightly above design capacity, or about 3% deficient; this is sufficiently close to consider the design satisfactory.

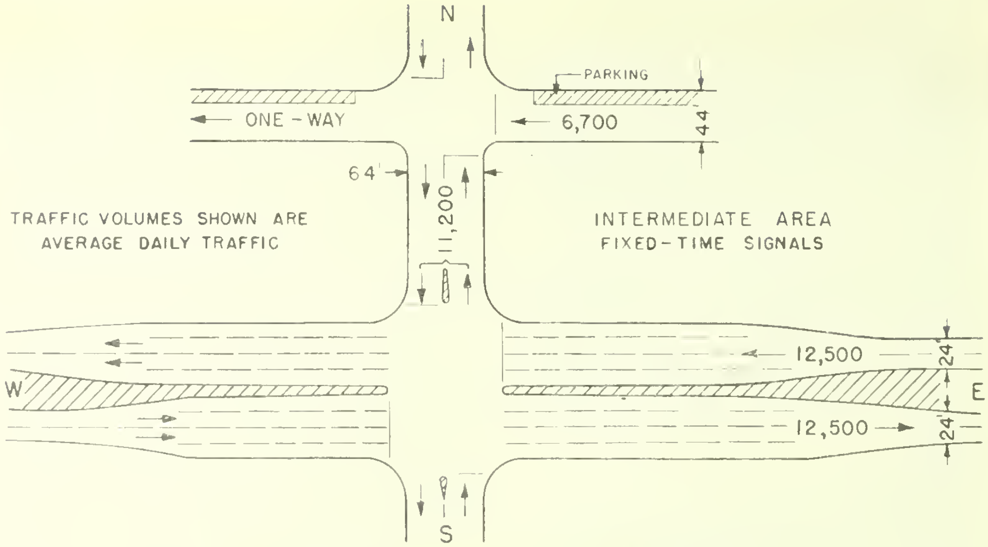

N
PARKING
ONE - WAY
6,700
44
64'
11,200
TRAFFIC VOLUMES SHOWN ARE AVERAGE DAILY TRAFFIC
INTERMEDIATE AREA FIXED-TIME SIGNALS
12,500
24
W
E
12,500
24
S

Figure 12.—Illustrative problems 43 and 44.

SOLUTION WITH AVERAGE DAILY TRAFFIC VOLUMES

In the first stages of preliminary design, when the pattern of highways and their intersections are being developed, street widths often must be tentatively established as a starting point. Only general traffic values are then available, customarily expressed in terms of average daily traffic volumes. Such information is of little value in final design of intersections, but in the absence of peak-hour volumes the average daily traffic volumes may be considered appropriate for preliminary design.

Where the design volumes are given in terms of average daily traffic it is necessary to convert them to peak-hour volumes in one direction of travel. Chart 15 gives this relation for average conditions. The peak-hour volume for design is assumed as 12 percent of the two-way average daily traffic, and as 55, 60, and 65 percent of the total peak-hour traffic as the predominant movement in one direction of travel for downtown, intermediate, and outlying areas, respectively. Chart 15 may be entered at the top with the total two-directional average daily traffic volume, or at the bottom with a one-way average daily traffic volume. Thus, the chart is applicable to both one- and two-way facilities. The vertical scale gives the design peak-hour volume in one direction of travel.

It will be noted that the assumption of the peak hour traffic as 12 percent of the two-way average daily traffic, and the use of three d values, results in different peak-hour percentages of the one-way average daily traffic, when the latter is one-half of the two-way daily volume. Thus, for one-way facilities, in chart 15, the resulting percentages that the peak-hour volume is of the one-way average daily traffic (obtained by dividing V by ½ ADT), are: downtown, 13.2; intermediate, 14.4; and outlying, 15.6.

Problem 43

Shown in figure 12 is a portion of a preliminary plan on which an east-west four-lane expressway is planned to cross at grade a north-south major street. The only traffic information available is in terms of average daily traffic, as indicated on the figure. The north-south street cannot be widened, but right-of-way permits widening of the approaches on the expressway. Determine the number of lanes required at the intersection in each direction on the expressway if the intersection is to operate at design capacity.

Solution: It is first necessary to convert the average daily traffic volume to peak-hour volume in one direction of travel. On the north-south street the two-directional daily volume of 11,200 vehicles in an intermediate area, from chart 15, is equivalent to a peak-hour volume in one direction of 800 vehicles; and on the expressway the daily volume of 12,500 vehicles in one direction corresponds to a peak-hour volume of 1,800 vehicles.

Using the north approach on the major street as approach A in chart 14, with $W/2=64/2=32$, two-way street with no parking in intermediate area, and an approach volume of 800 v.p.h., a value of $G/C=0.46$ is obtained. From this point a straight line is projected to the right through the zero point of line y-y, intersecting a value of $G/C=0.44$ for approach B. From this point proceed to the right to an approach volume of 1,800 v.p.h., then up to expressway with widened approaches, and read at the right a required approach width of about 50 feet.

The two-lane pavement in each direction on the expressway should therefore be widened in advance of and beyond the intersection to $50/12=4.2$, or four lanes. The length of widened pavements should be of sufficient length, as noted under the sketch in chart 13.

Problem 44

In figure 12, the north-south major street intersects a one-way street at a point several hundred feet north of the expressway. According to the average daily traffic volumes indicated and street widths provided, determine whether adequate capacity is available at this intersection.

Solution: The left portion of chart 14 is used for one approach on the north-south street, as in problem 43. The right portion of the chart is used for the east approach of the one-way street with $W/2=44$, one-way street with parking in intermediate area, and an approach volume of 960 v.p.h. (derived from chart 15 as the equivalent of an average daily traffic in one direction of 6,700). A straight line between the two resulting values of G/C indicates, at the intersection of the y-y line, that more than adequate capacity is provided.

LIMITATIONS IN USE OF CHARTS 14 AND 15

Although charts 14 and 15 are convenient tools for preliminary design, the limitations in their use should be recognized:

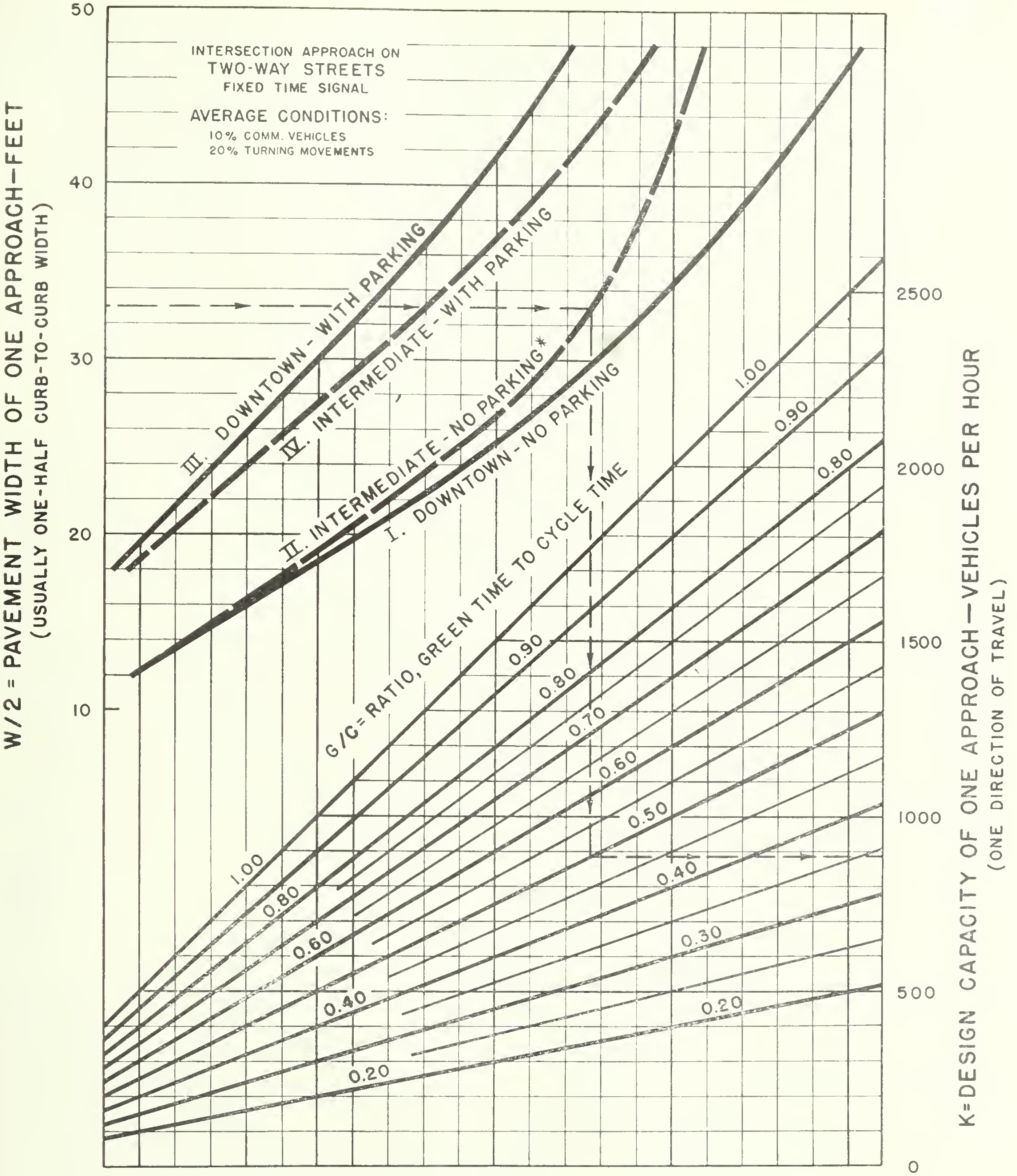
1. Results are approximate, since average conditions are assumed.

2. On two-way facilities it is assumed that the volume of left-turning vehicles can be accommodated without requiring three-phase signal control. Generally on major facilities not more than 80 to 120 left-turning vehicles per hour can be handled without a separate signal indication. Even in preliminary design, if it is known that the left turns on one approach will exceed about 100 vehicles per hour, the more detailed charts should be used.

3. Where average daily traffic volumes are used as the traffic basis, and corresponding one-way peak-hour volumes are taken from chart 15, it should be remembered that the peak-hour volumes on each approach may not occur simultaneously. At intersections involving two-way streets on which the signal timing remains the same throughout the day, the peak-hour volumes thus obtained and used may be appropriate. However, if the signal timing is different during the morning peak from that during the evening peak, because of load distribution, or if one or both of the intersecting streets are one-way, the peak-hour volumes based on average daily traffic may not be representative. If the designer has some knowledge of the peak distributions, he can make adjustments in the hourly approach volumes to arrive at the information needed for both the morning and evening peak hours, one or both of which may govern the design; if not, the direct use of chart 15 for each approach will give results in design on the safe side.

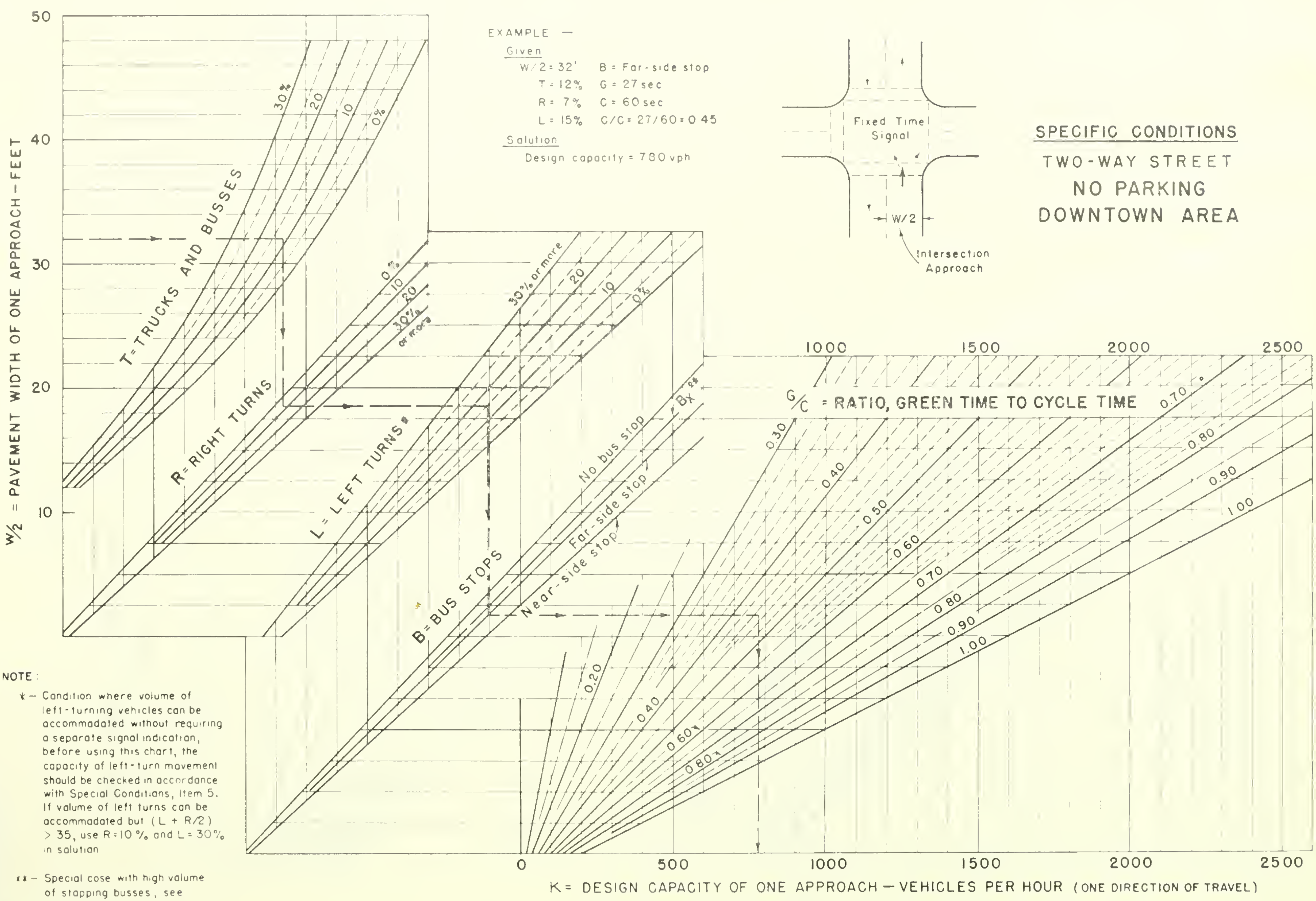
4. Chart 14 may not always apply to those intersections where a given minimum green interval must be maintained, as for a pedestrian crossing. However, the values of G/C obtained in the solution for any intersection can be tested with logical cycle lengths to determine whether the required green interval may be obtained.

5. For approximate results, chart 14 may also be used for T or Y intersections.



* Also applicable to outlying or rural areas.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
TWO-WAY STREETS—AVERAGE CONDITIONS
CHART 1



EXAMPLE —

Given

W/2 = 32'   B = Far-side stop

T = 12%   G = 27 sec

R = 7%   C = 60 sec

L = 15%   G/C = 27/60 = 0.45

Solution

Design capacity = 780 vph

SPECIFIC CONDITIONS

TWO-WAY STREET

NO PARKING

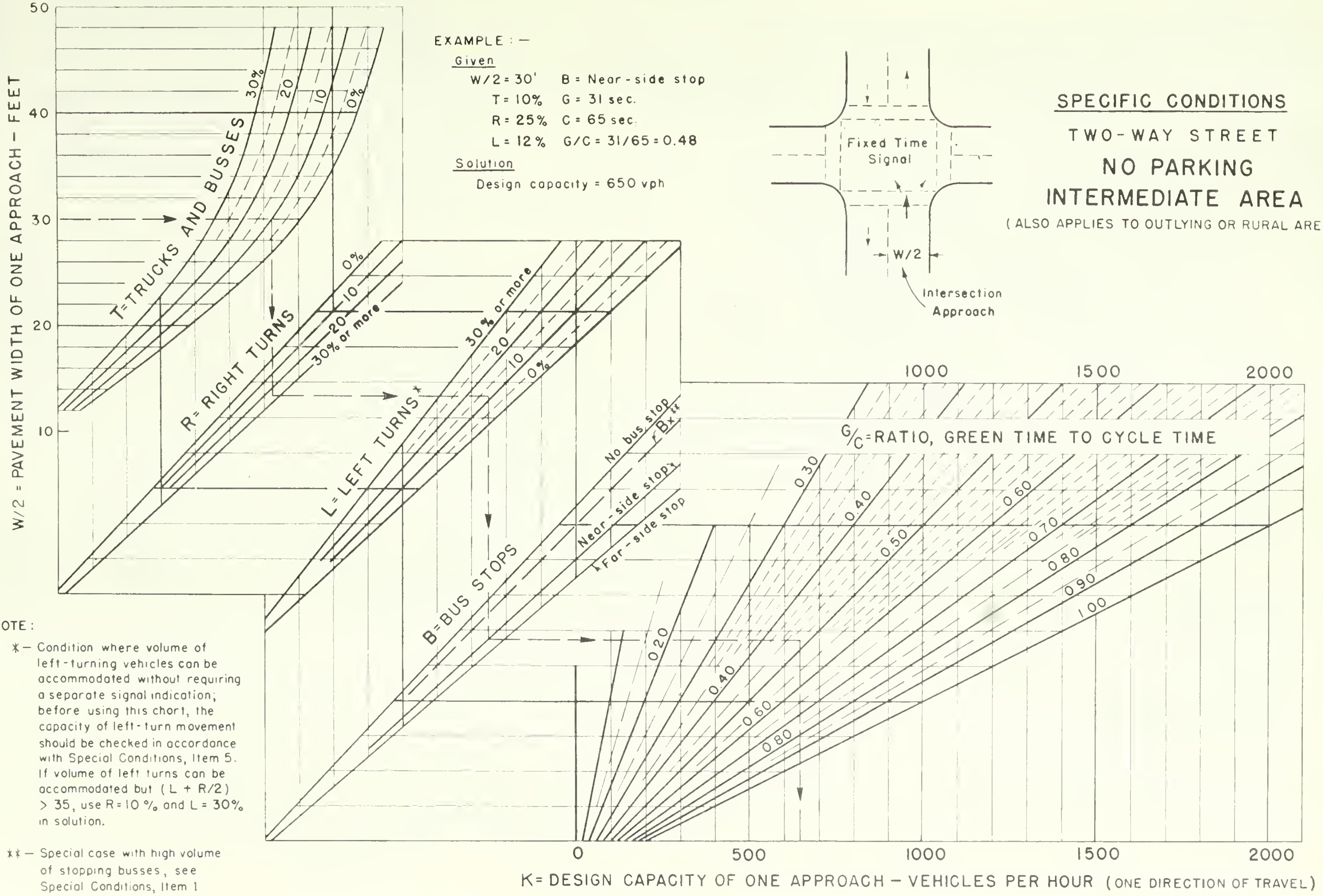
DOWNTOWN AREA

NOTE:

* — Condition where volume of left-turning vehicles can be accommodated without requiring a separate signal indication, before using this chart, the capacity of left-turn movement should be checked in accordance with Special Conditions, Item 5. If volume of left turns can be accommodated but (L + R/2) > 35, use R = 10% and L = 30% in solution

** — Special case with high volume of stopping busses, see Special Conditions, Item 1

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
TWO-WAY STREET — NO PARKING, DOWNTOWN AREA
CHART 2



EXAMPLE: —

Given

W/2 = 30' B = Near-side stop
T = 10% G = 31 sec.
R = 25% C = 65 sec.
L = 12% G/C = 31/65 = 0.48

Solution

Design capacity = 650 vph

SPECIFIC CONDITIONS

TWO-WAY STREET

NO PARKING

INTERMEDIATE AREA

(ALSO APPLIES TO OUTLYING OR RURAL AREAS)

NOTE:

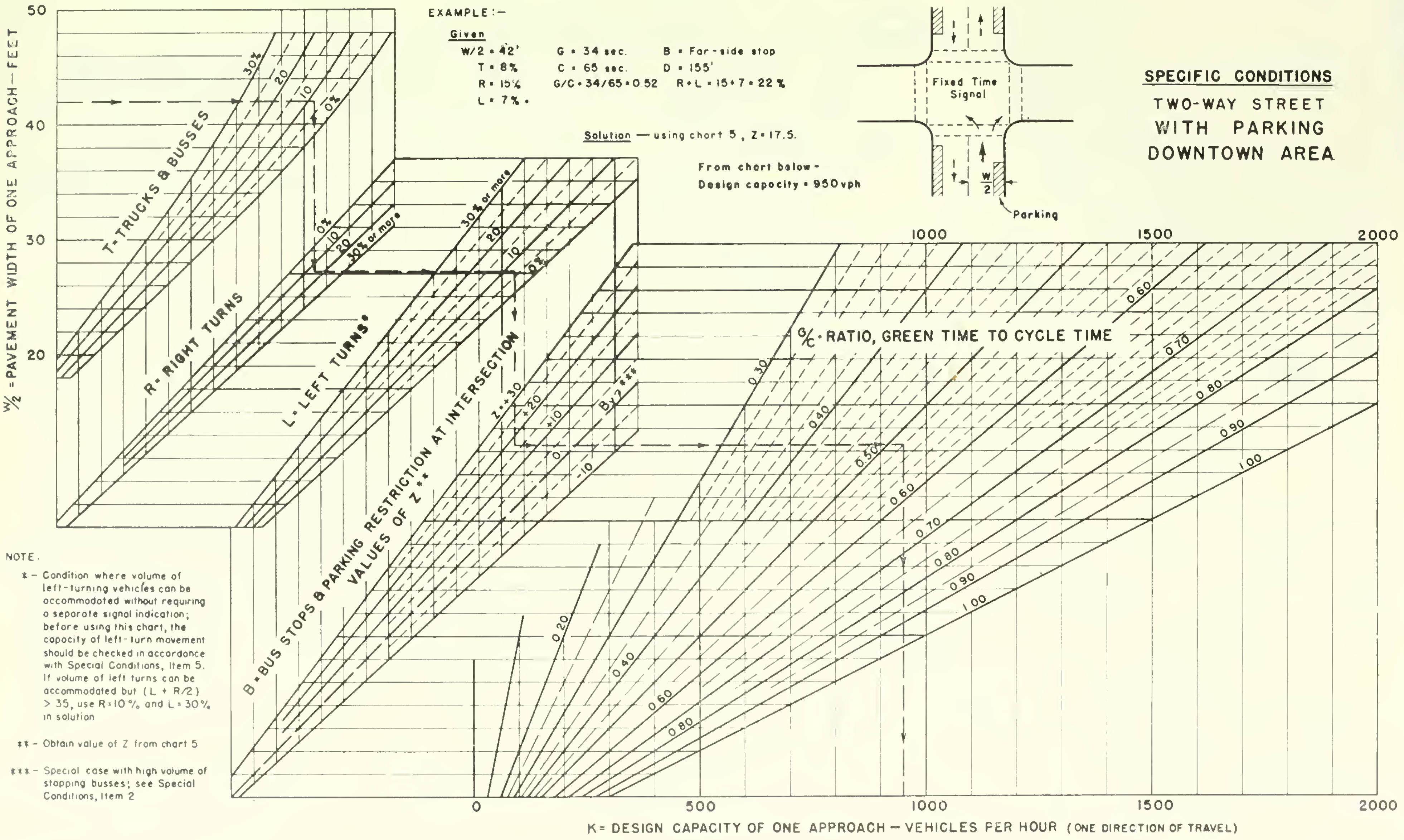
* — Condition where volume of left-turning vehicles can be accommodated without requiring a separate signal indication; before using this chart, the capacity of left-turn movement should be checked in accordance with Special Conditions, Item 5. If volume of left turns can be accommodated but (L + R/2) > 35, use R = 10% and L = 30% in solution.

** — Special case with high volume of stopping busses, see Special Conditions, Item 1

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

TWO-WAY STREET — NO PARKING, INTERMEDIATE AREA

CHART 3



EXAMPLE:—

Given

W/2 = 42'   G = 34 sec.   B = Far-side stop
T = 8%   C = 65 sec.   D = 155'
R = 15%   G/C = 34/65 = 0.52   R + L = 15 + 7 = 22%
L = 7%.

Solution — using chart 5, Z = 17.5.

From chart below —
Design capacity = 950 vph

SPECIFIC CONDITIONS

TWO-WAY STREET
WITH PARKING
DOWNTOWN AREA

NOTE.

* — Condition where volume of left-turning vehicles can be accommodated without requiring a separate signal indication; before using this chart, the capacity of left-turn movement should be checked in accordance with Special Conditions, Item 5. If volume of left turns can be accommodated but (L + R/2) > 35, use R = 10%, and L = 30% in solution

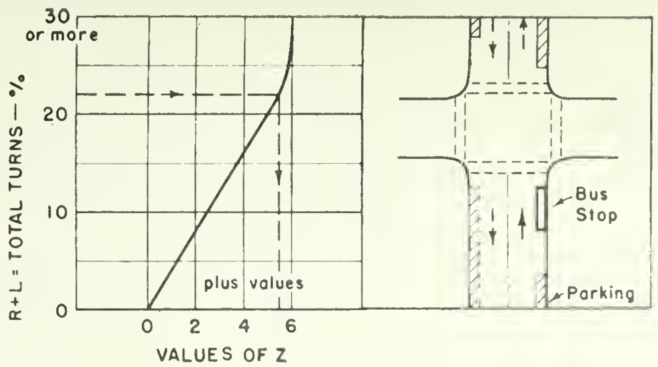
** — Obtain value of Z from chart 5

*** — Special case with high volume of stopping busses; see Special Conditions, Item 2

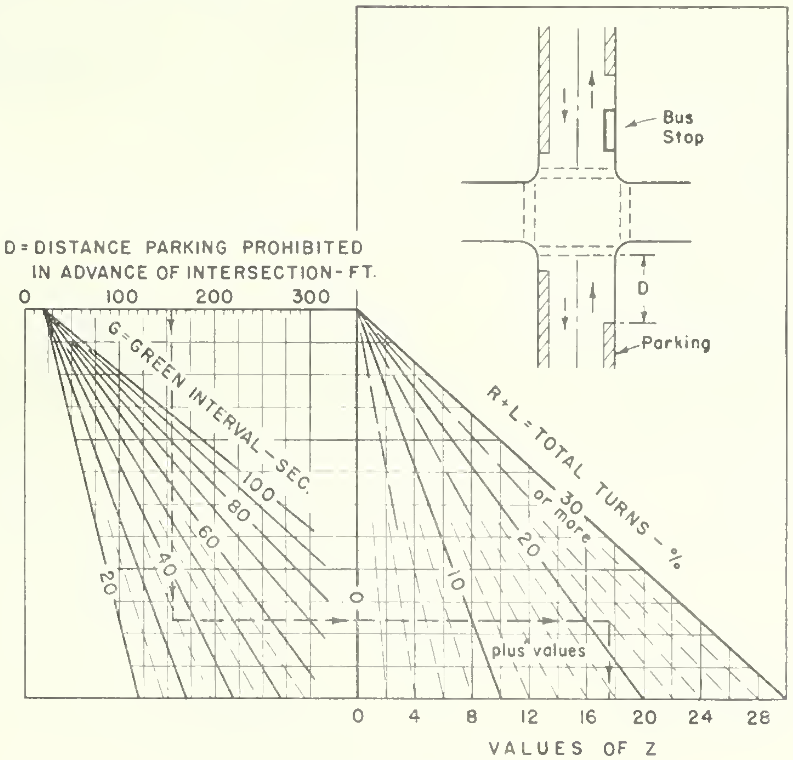
DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

TWO-WAY STREET — WITH PARKING, DOWNTOWN AREA

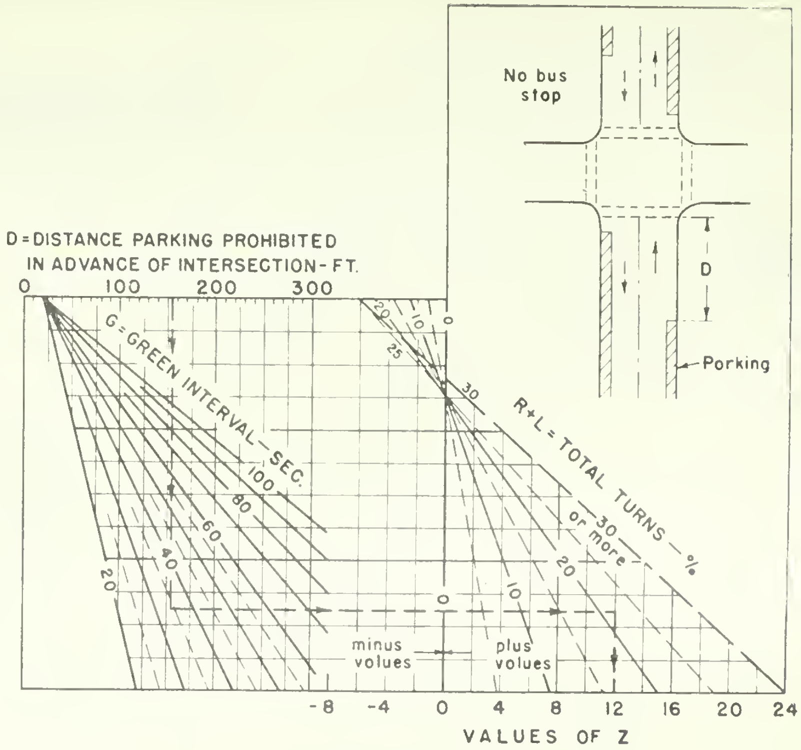
CHART 4



NEAR-SIDE BUS STOP
CHART A



FAR-SIDE BUS STOP
CHART B

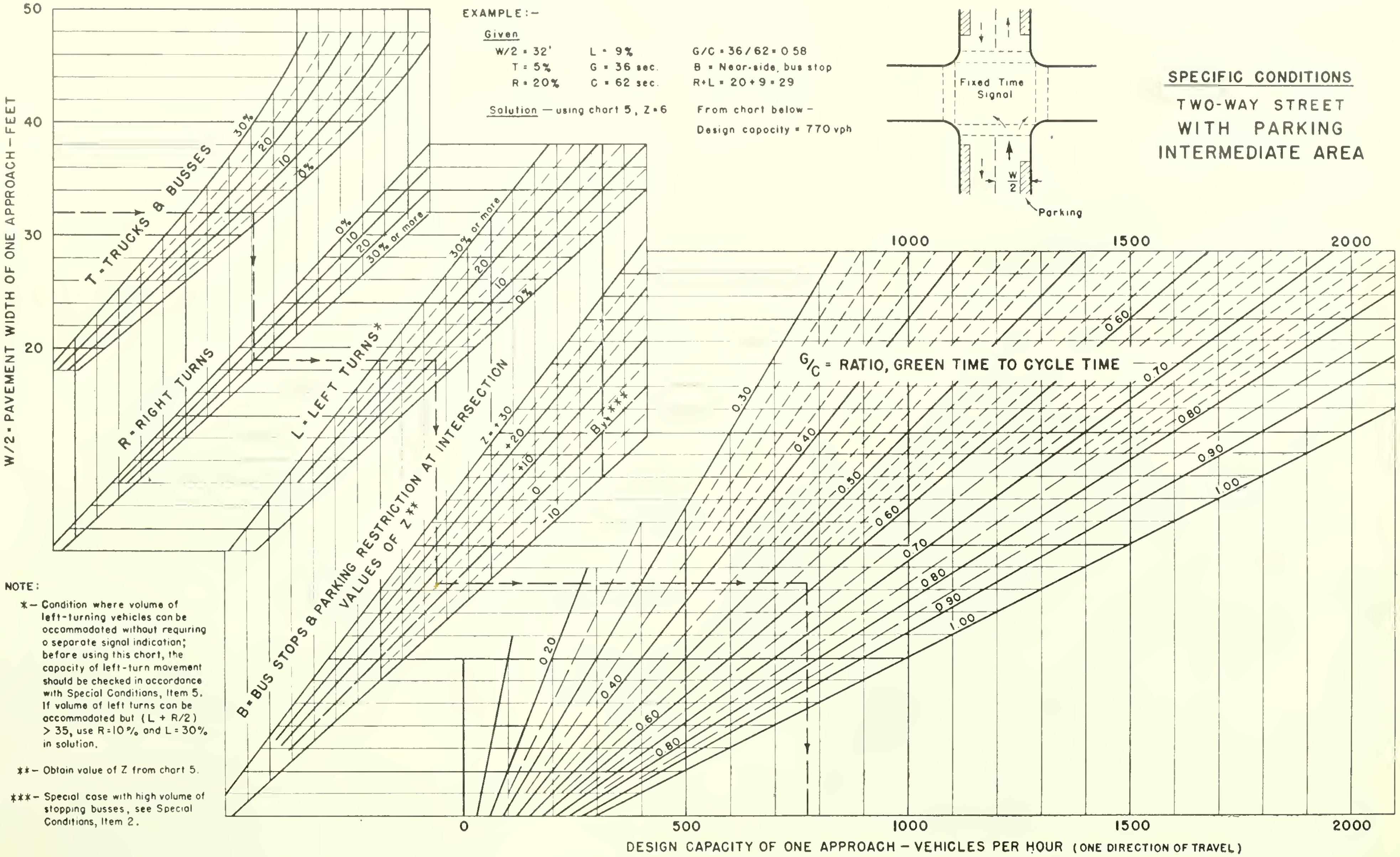


NO BUS STOP
CHART C

NOTE:
Z = COMBINED CORRECTION FOR BUS STOP AND PARKING RESTRICTION IN PERCENT, FOR USE IN CHARTS 4 AND 6.

WHERE PARKING IS PROHIBITED FOR A LIMITED DISTANCE IN BOTH DIRECTIONS FROM THE CROSS STREET, SEE SPECIAL CONDITIONS, ITEM 4

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
SUPPLEMENT TO CHARTS 4 AND 6
CHART 5



EXAMPLE:–

Given

W/2 = 32'
T = 5%
R = 20%
L = 9%
G = 36 sec.
C = 62 sec.
G/C = 36/62 = 0.58
B = Near-side, bus stop
R+L = 20+9 = 29

Solution — using chart 5, Z = 6

From chart below –
Design capacity = 770 vph

SPECIFIC CONDITIONS

TWO-WAY STREET
WITH PARKING
INTERMEDIATE AREA

NOTE:

* – Condition where volume of left-turning vehicles can be accommodated without requiring a separate signal indication; before using this chart, the capacity of left-turn movement should be checked in accordance with Special Conditions, Item 5. If volume of left turns can be accommodated but (L + R/2) > 35, use R = 10% and L = 30% in solution.

** – Obtain value of Z from chart 5.

*** – Special case with high volume of stopping busses, see Special Conditions, Item 2.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
TWO-WAY STREET—WITH PARKING, INTERMEDIATE AREA
CHART 6

TWO-WAY STREET
WITH RIGHT-TURN LANE
NO SEPARATE SIGNAL INDICATION FOR TURNING MOVEMENT

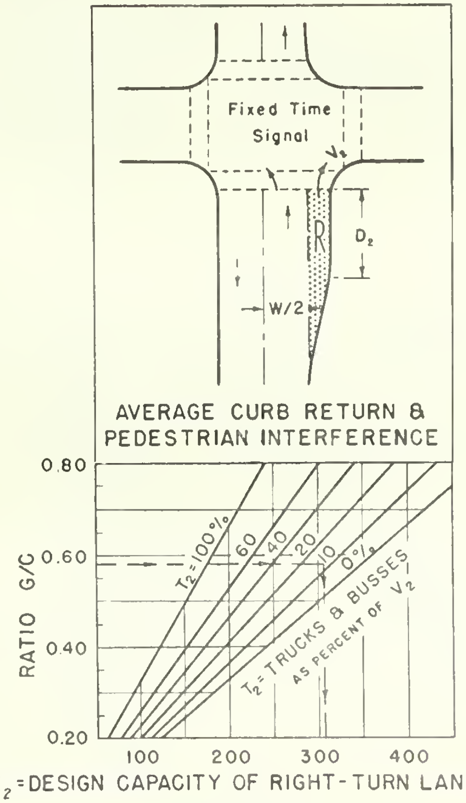


CHART A

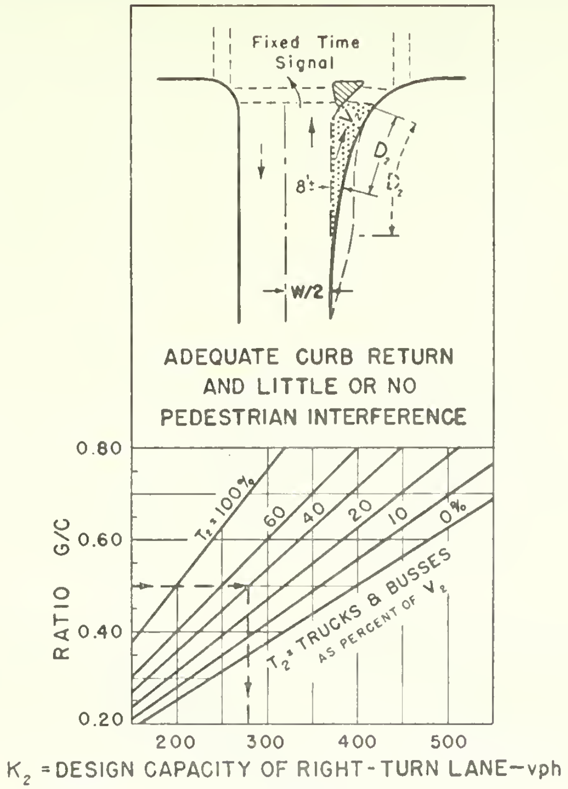


CHART B

NOTE FOR CHART C:

LENGTH OF RIGHT-TURN LANE BASED ON THE AVERAGE NUMBER OF VEHICLES THAT WOULD STORE PER CYCLE, FOR V_2 SHOWN.

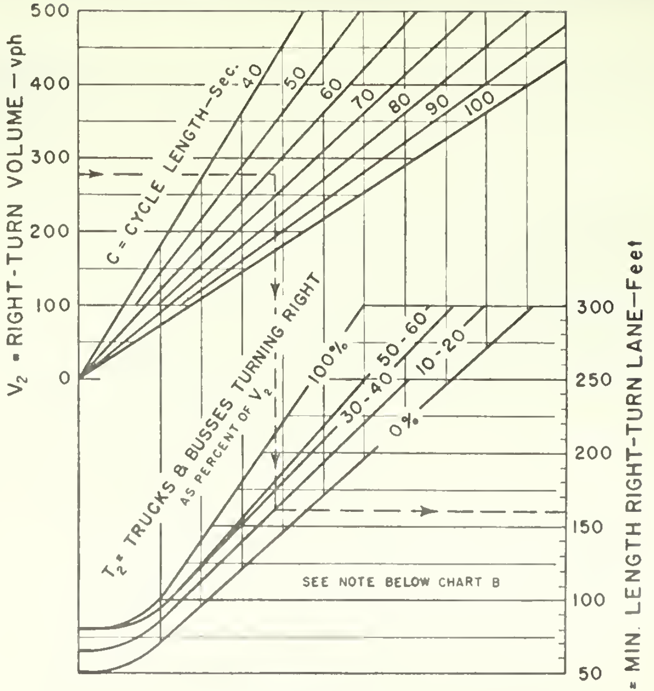


CHART C

DESIGN CAPACITY OF ONE APPROACH:—

(1) OBTAIN K_2 FROM CHART A OR B.

(2) OBTAIN DESIGN CAPACITY OF COMBINED THROUGH AND LEFT-TURN MOVEMENT (M_2) FROM CHART 2 OR 3 USING:

W/2 = APPROACH WIDTH EXCLUSIVE OF RIGHT-TURN LANE; AND

R = 0%.

(3) CALCULATE $V_2 = \frac{M_2 \times R}{100 - R}$

(4) WHEN $V_2 < K_2$: $K = M_2 + V_2$

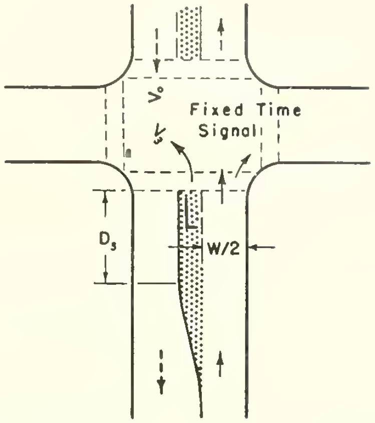
(5) WHEN $V_2 > K_2$: FIND $M_2' = \frac{K_2(100-R)}{R}$ AND $K = M_2' + K_2$

(6) OBTAIN D_2 FROM CHART C FOR V_2 OR K_2, WHICHEVER GOVERNS.

NOTE: POSSIBLE CAPACITY OF RIGHT-TURN LANE = $1.2 K_2$.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
TWO-WAY STREET — WITH RIGHT-TURN LANE
NO SEPARATE SIGNAL INDICATION FOR TURNING MOVEMENT
CHART 7

TWO-WAY STREET WITH LEFT-TURN LANE NO SEPARATE SIGNAL INDICATION FOR TURNING MOVEMENT



DESIGN CAPACITY OF ONE APPROACH:—

(1) OBTAIN K_3 FROM CHART A OR B

(2) OBTAIN DESIGN CAPACITY OF COMBINED THROUGH AND RIGHT-TURN MOVEMENT (M_3) FROM CHART 2, 3, 4, OR 6, USING:

W/2 = APPROACH WIDTH EXCLUSIVE OF LEFT-TURN LANE; AND

L = 0%

(3) CALCULATE $V_3 = \frac{M_3 \times L}{100-L}$

(4) WHEN $V_3 < K_3$: $K = M_3 + V_3$

(5) WHEN $V_3 > K_3$: FIND $M_3' = \frac{K_3(100-L)}{L}$ AND $K = M_3' + K_3$

(6) OBTAIN D_3 FROM CHART C FOR V_3 OR K_3, WHICHEVER GOVERNS

NOTE: POSSIBLE CAPACITY OF LEFT-TURN LANE = $1.2K_3$

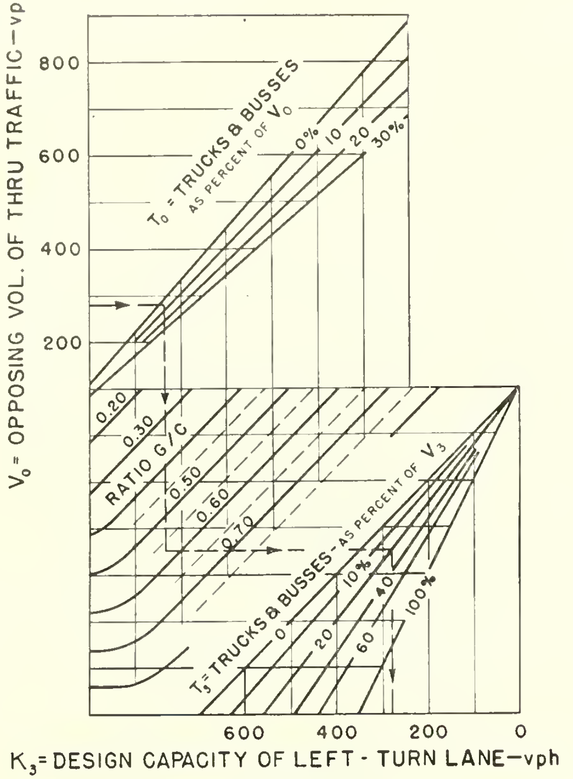


CHART A

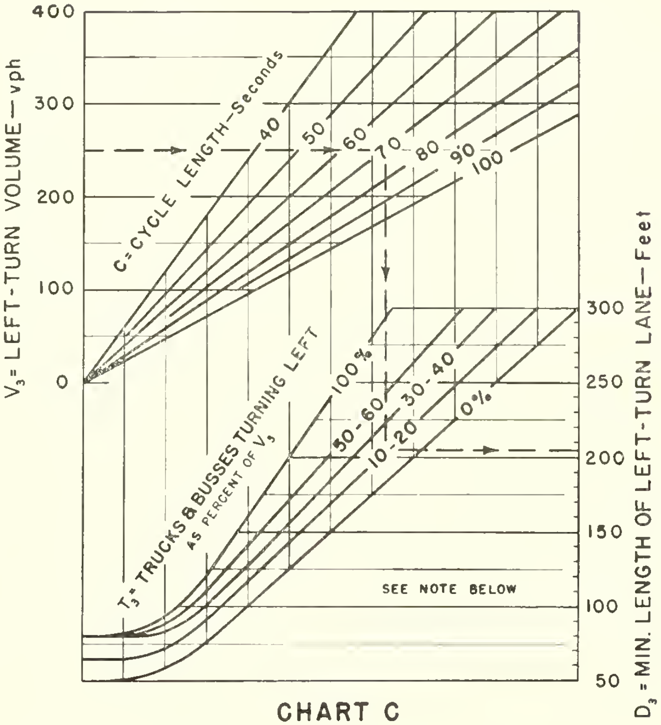
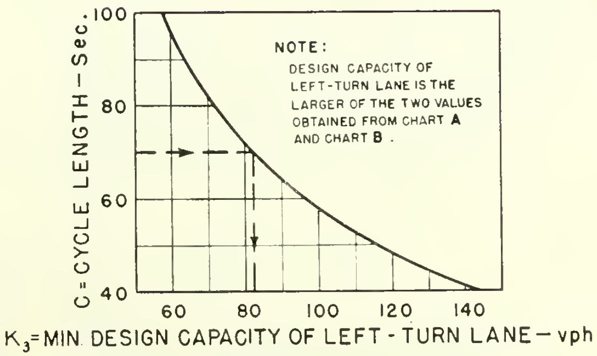


CHART C

NOTE FOR CHART C:
LENGTH OF LEFT-TURN LANE BASED ON 1.5 TIMES THE AVERAGE NUMBER OF VEHICLES THAT WOULD STORE PER CYCLE, FOR V_3 SHOWN.



NOTE:
DESIGN CAPACITY OF LEFT-TURN LANE IS THE LARGER OF THE TWO VALUES OBTAINED FROM CHART A AND CHART B.

CHART B

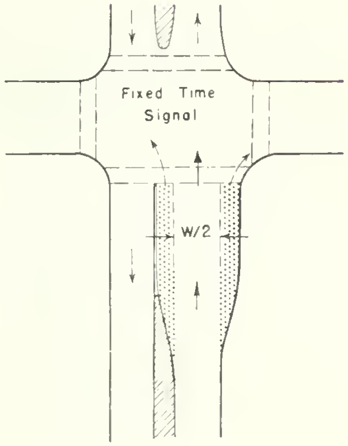
DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
TWO-WAY STREET — WITH LEFT-TURN LANE
NO SEPARATE SIGNAL INDICATION FOR TURNING MOVEMENT
CHART 8

TWO-WAY STREET

WITH LEFT- AND RIGHT-TURN LANES

NO SEPARATE SIGNAL INDICATION

FOR TURNING MOVEMENTS



DESIGN CAPACITY OF ONE APPROACH:-

(1) OBTAIN K_2 FROM CHART 7 AND K_3 FROM CHART 8

(2) OBTAIN DESIGN CAPACITY OF THROUGH MOVEMENT (M_1), FROM CHART 2 OR 3, USING:

W/2 = APPROACH WIDTH EXCLUSIVE OF TURNING LANES, R = 0 %; AND L = 0 %

(3) CALCULATE $V_2 = \frac{M_1 \times R}{100-R-L}$ AND $V_3 = \frac{M_1 \times L}{100-R-L}$

(4) WHEN $V_2 < K_2$ AND $V_3 < K_3$: $K = M_1 + V_2 + V_3$

(5) WHEN $V_2 > K_2$ AND/OR $V_3 > K_3$, CALCULATE M_1' ON BASIS OF K_2 OR K_3, WHICHEVER GOVERNS; IF K_3 GOVERNS, $M_1' = \frac{K_3(100-R-L)}{L}$, AND $V_2' = \frac{M_1' R}{100-R-L}$, AND $K = M_1' + V_2' + K_3$

(6) OBTAIN D_2 AND D_3 FROM CHARTS 7 AND 8.

NOTE: FOR POSSIBLE CAPACITY OF RIGHT- OR LEFT-TURN LANE MULTIPLY BY 1.2

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

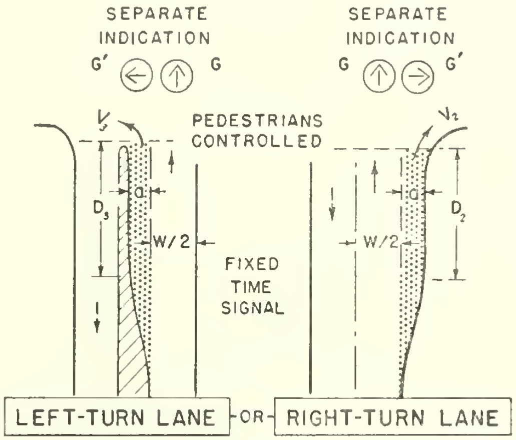
TWO-WAY STREET — WITH LEFT- AND RIGHT-TURN LANES

NO SEPARATE SIGNAL INDICATION FOR TURNING MOVEMENTS

CHART 9

TWO-WAY STREET
WITH RIGHT- OR LEFT-TURN LANE

SEPARATE SIGNAL INDICATION
FOR RIGHT- OR LEFT-TURN MOVEMENT



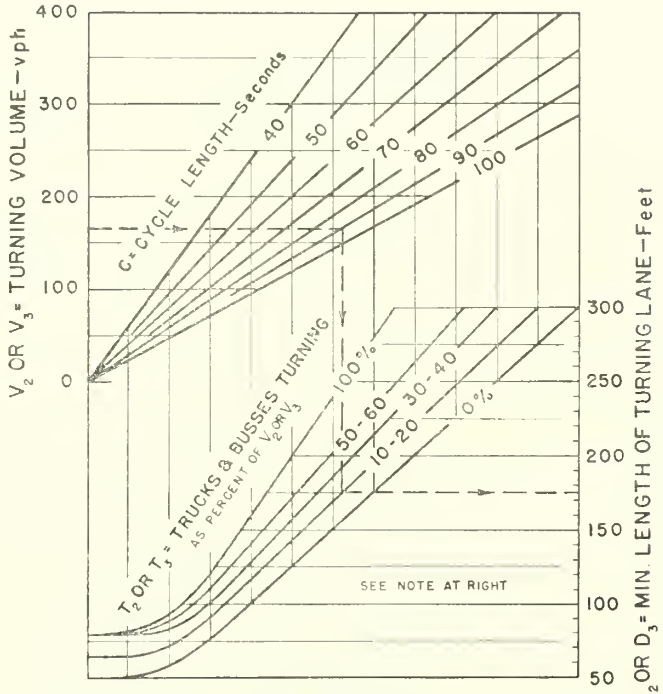


CHART B

NOTE FOR CHART B:
LENGTH OF RIGHT- OR LEFT-TURN LANE BASED ON 1.5 TIMES THE AVERAGE NUMBER OF VEHICLES THAT WOULD STORE PER CYCLE, FOR V_2 OR V_3 SHOWN.

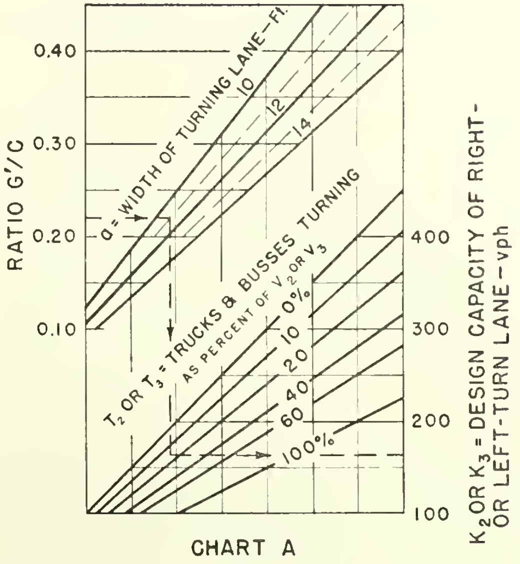


CHART A

DESIGN CAPACITY OF ONE APPROACH:

WITH LEFT-TURN LANE

(1) OBTAIN K_3 FROM CHART A.

(2) OBTAIN DESIGN CAPACITY OF COMBINED THROUGH AND RIGHT-TURN MOVEMENT (M_3) FROM CHART 2, 3, 4, OR 6, USING:

W/2 = APPROACH WIDTH EXCLUSIVE OF LEFT-TURN LANE; AND

L = 0%.

(3) CALCULATE $V_3 = \frac{M_3 \times L}{100 - L}$

(4) WHEN $V_3 < K_3$: $K = M_3 + V_3$

(5) WHEN $V_3 > K_3$: FIND $M_3' = \frac{K_3(100-L)}{L}$ AND $K = M_3' + K_3$

(6) OBTAIN D_3 FROM CHART B FOR V_3 OR K_3, WHICHEVER GOVERNS.

NOTE: POSSIBLE CAPACITY OF LEFT-TURN LANE = $1.2K_3$

WITH RIGHT-TURN LANE

(1) OBTAIN K_2 FROM CHART A.

(2) OBTAIN DESIGN CAPACITY OF COMBINED THROUGH AND LEFT-TURN MOVEMENT (M_2) FROM CHART 2 OR 3, USING:

W/2 = APPROACH WIDTH EXCLUSIVE OF RIGHT-TURN LANE; AND

R = 0%.

(3) CALCULATE $V_2 = \frac{M_2 \times R}{100 - R}$

(4) WHEN $V_2 < K_2$: $K = M_2 + V_2$

(5) WHEN $V_2 > K_2$: FIND $M_2' = \frac{K_2(100-R)}{R}$ AND $K = M_2' + K_2$

(6) OBTAIN D_2 FROM CHART B FOR V_2 OR K_2, WHICHEVER GOVERNS.

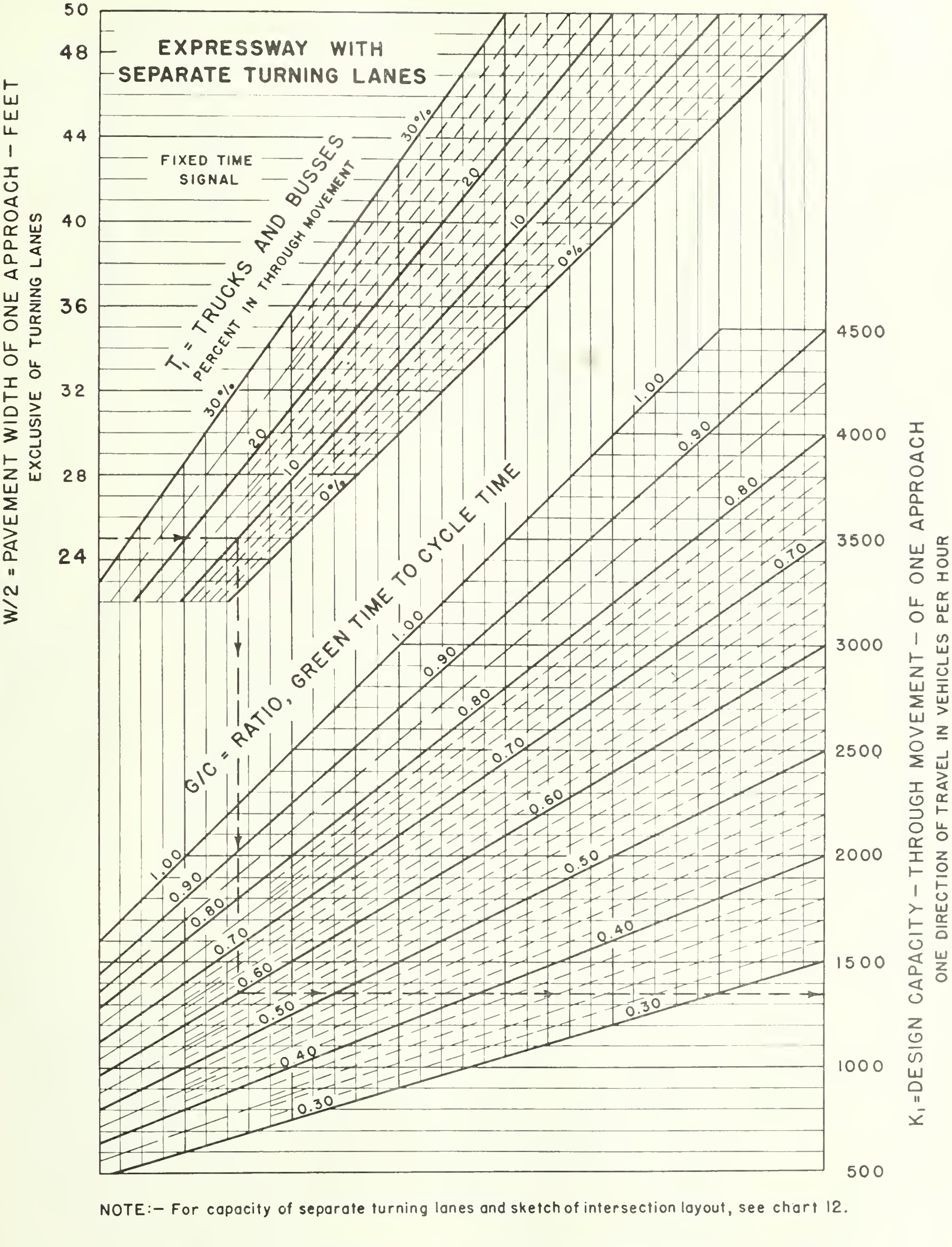
NOTE: POSSIBLE CAPACITY OF RIGHT-TURN LANE = $1.2K_2$

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

TWO-WAY STREET—WITH RIGHT- OR LEFT-TURN LANE

SEPARATE SIGNAL INDICATION FOR TURNING MOVEMENT

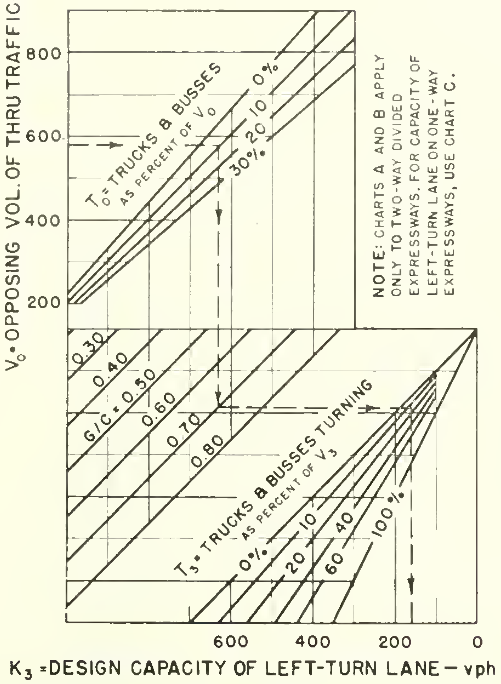
CHART 10



NOTE:– For capacity of separate turning lanes and sketch of intersection layout, see chart 12.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
EXPRESSWAYS WITH SEPARATE TURNING LANES
CAPACITY OF THROUGH LANES ONLY

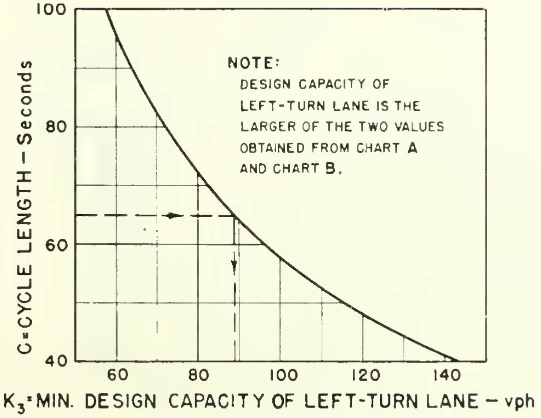
CHART 11



NOTE: CHARTS A AND B APPLY ONLY TO TWO-WAY DIVIDED EXPRESSWAYS. FOR CAPACITY OF LEFT-TURN LANE ON ONE-WAY EXPRESSWAYS, USE CHART C.

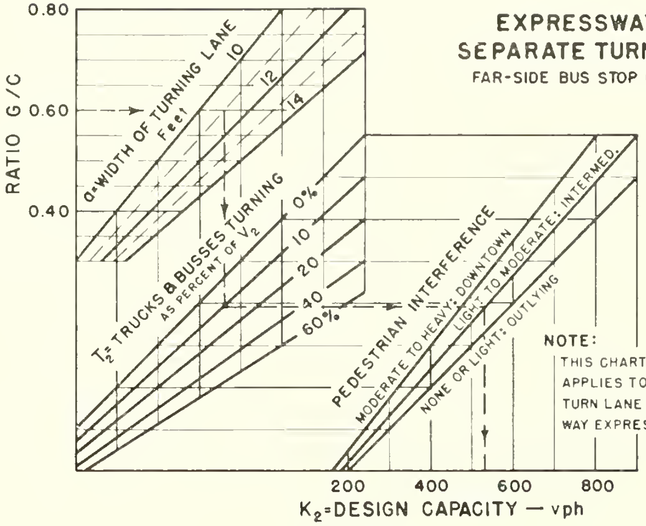
CHART A

CHART B



NOTE:
DESIGN CAPACITY OF LEFT-TURN LANE IS THE LARGER OF THE TWO VALUES OBTAINED FROM CHART A AND CHART B.

CHARTS A & B: CAPACITY OF LEFT-TURN LANE, NO SEPARATE SIGNAL INDICATION



NOTE:
THIS CHART ALSO APPLIES TO LEFT-TURN LANE ON ONE-WAY EXPRESSWAY.

CHART C

CAPACITY OF RIGHT-TURN LANE NO SEPARATE SIGNAL INDICATION

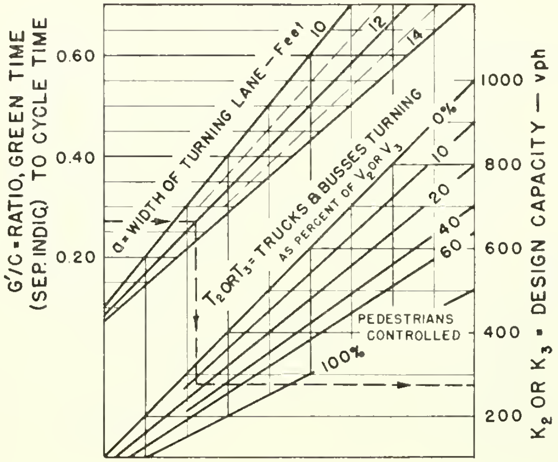
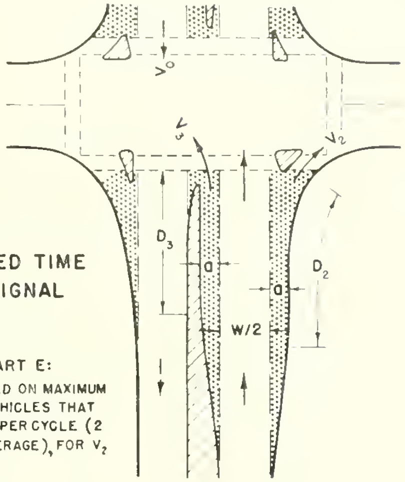


CHART D

CAPACITY OF RIGHT- OR LEFT-TURN LANE WITH SEPARATE SIGNAL INDICATION

EXPRESSWAY WITH SEPARATE TURNING LANES

FAR-SIDE BUS STOP OR NO BUS STOP



FIXED TIME SIGNAL

NOTE FOR CHART E:
LENGTH BASED ON MAXIMUM NUMBER OF VEHICLES THAT WOULD STORE PER CYCLE (2 TIMES THE AVERAGE), FOR V₂ OR V₃ SHOWN.

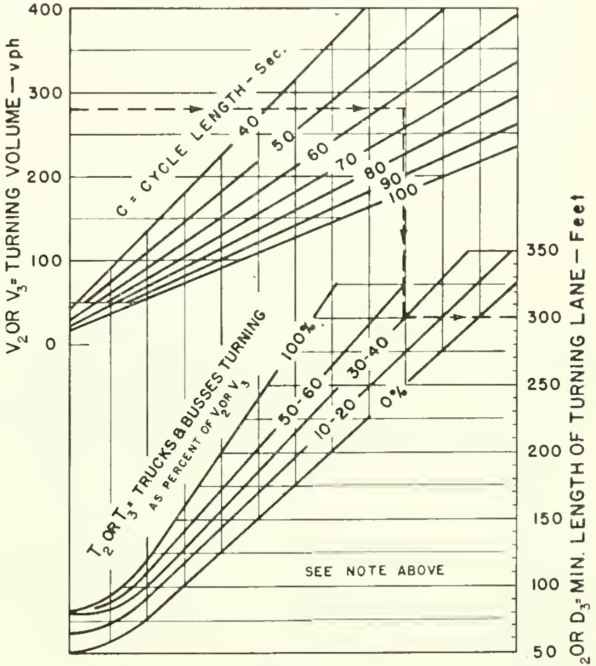


CHART E

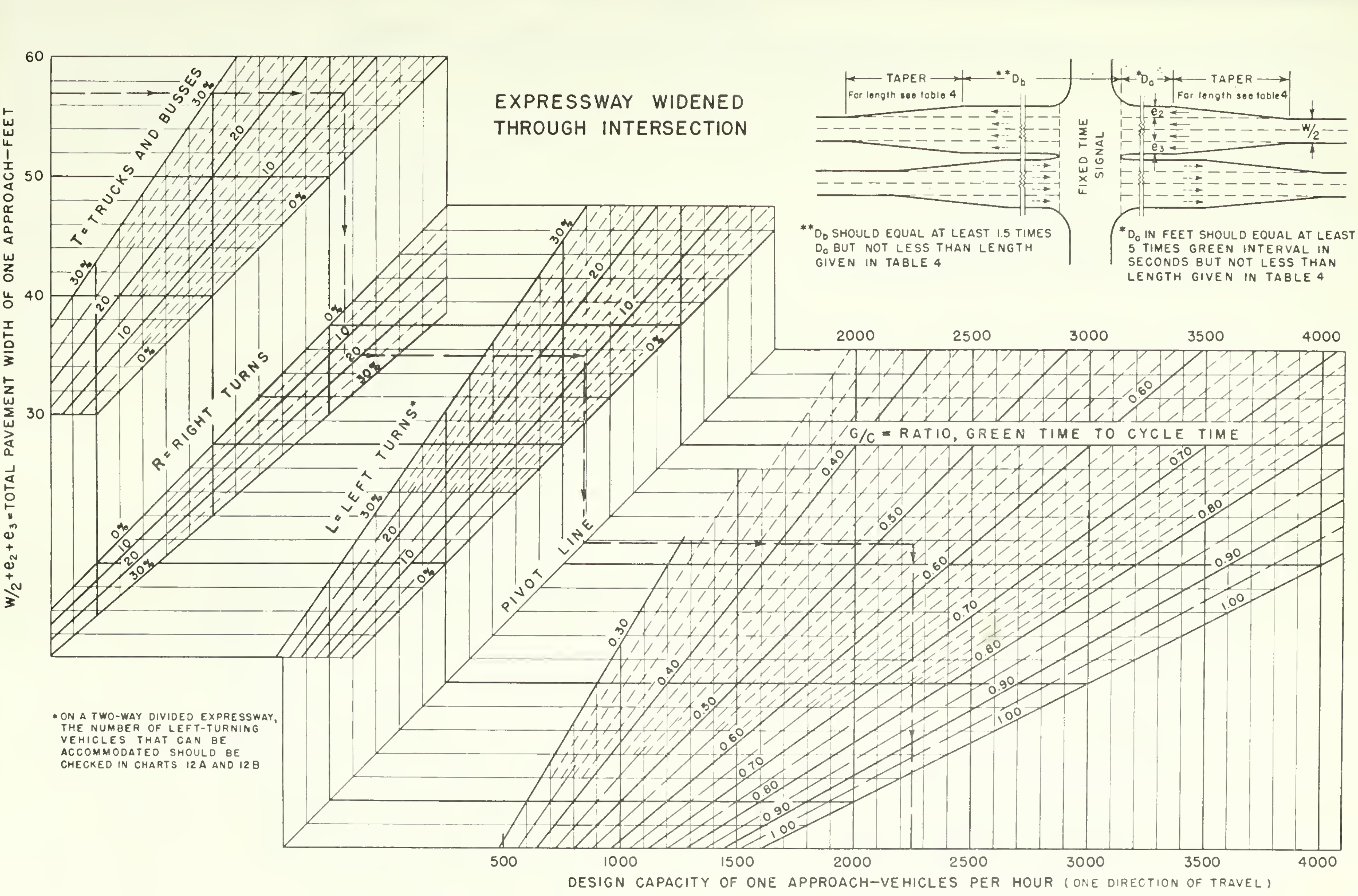
LENGTH OF RIGHT- OR LEFT-TURN LANE WITH OR WITHOUT SEPARATE SIGNAL INDICATION

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

EXPRESSWAYS WITH SEPARATE TURNING LANES

CAPACITY OF SEPARATE TURNING LANES ONLY

CHART 12



**Db SHOULD EQUAL AT LEAST 1.5 TIMES Da BUT NOT LESS THAN LENGTH GIVEN IN TABLE 4

*Da IN FEET SHOULD EQUAL AT LEAST 5 TIMES GREEN INTERVAL IN SECONDS BUT NOT LESS THAN LENGTH GIVEN IN TABLE 4

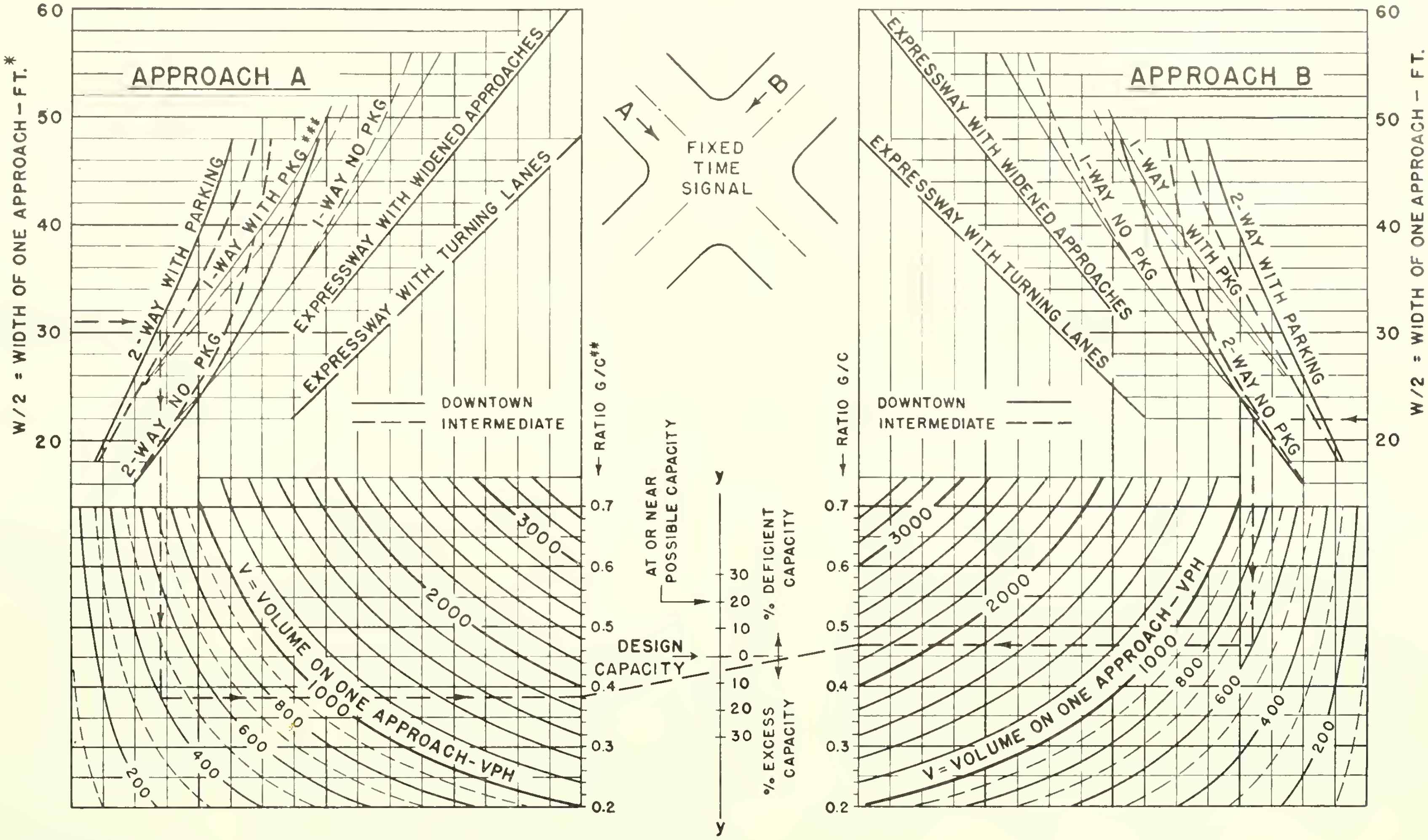
*ON A TWO-WAY DIVIDED EXPRESSWAY, THE NUMBER OF LEFT-TURNING VEHICLES THAT CAN BE ACCOMMODATED SHOULD BE CHECKED IN CHARTS 12A AND 12B

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

EXPRESSWAY WIDENED THROUGH INTERSECTION

CHART 13

OVER-ALL INTERSECTION CAPACITY — AVERAGE CONDITIONS



* — FOR TWO-WAY STREET —

W/2 IS WIDTH OF ONE APPR., USUALLY 1/2 NORMAL CURB-TO-CURB WIDTH.

FOR ONE-WAY STREET —

W/2 IS WIDTH OF STREET, USUALLY NORMAL CURB-TO-CURB WIDTH.

FOR EXPRESSWAY WITH TURNING LANES —

W/2 IS NORMAL WIDTH OF TRAVELED WAY EXCLUSIVE OF TURNING LANES.

FOR EXPRESSWAY WITH WIDENED APPROACHES —

W/2 IS THE FULL WIDTH OF WIDENED APPROACH, INCLUDING THE ADDED LANES (AND CORRESPONDS TO $W/2 + e_2 + e_3$ IN CHART 13).

** — PROPORTION OF GREEN TIME REQUIRED ON THE ONE APPROACH FOR OPERATION AT DESIGN CAPACITY.

*** — PARKING ON ONE SIDE ONLY.

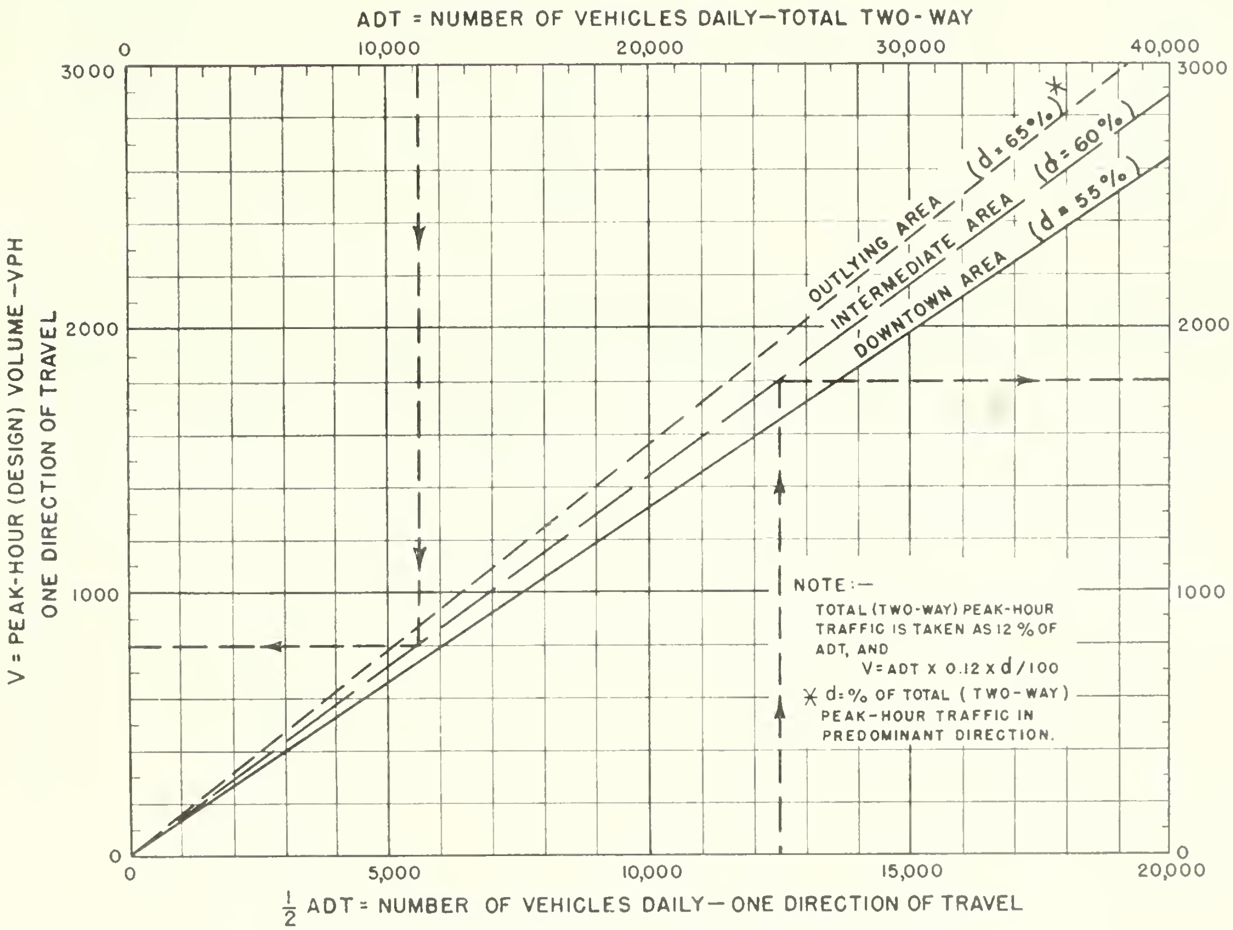
AVERAGE CONDITIONS — EACH APPROACH:

TRUCKS AND BUSSES = 10%
TURNING MOVEMENTS —
STREETS = 20%
EXPRESSWAYS = 15%
AMBER = 10% OF CYCLE

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

OVER-ALL INTERSECTION CAPACITY — AVERAGE CONDITIONS

CHART 14



RELATION BETWEEN HOURLY AND DAILY TRAFFIC
CHART 15

U. S. GOVERNMENT PRINTING OFFICE: 1951—911329

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

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(See also adjacent column)

Reports of the Chief of the Bureau of Public Roads:
1937, 10 cents. 1938, 10 cents. 1939, 10 cents.

Work of the Public Roads Administration:
1940, 10 cents. 1942, 10 cents. 1948, 20 cents.
1941, 15 cents. 1946, 20 cents. 1949, 25 cents.
1947, 20 cents.

Annual Report, Bureau of Public Roads, 1950, 25 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.
Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.
Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.
Part 4 . . . Official Inspection of Vehicles. 10 cents.
Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.
Part 6 . . . The Accident-Prone Driver. 10 cents.

UNIFORM VEHICLE CODE

Act I.—Uniform Motor-Vehicle Administration, Registration, Certificate of Title, and Antitheft Act. 10 cents.
Act II.—Uniform Motor-Vehicle Operators' and Chauffeurs' License Act. 10 cents.
Act III.—Uniform Motor-Vehicle Civil Liability Act. 10 cents.
Act IV.—Uniform Motor-Vehicle Safety Responsibility Act. 10 cents.
Act V.—Uniform Act Regulating Traffic on Highways. 20 cents.
Model Traffic Ordinance. 15 cents.

MISCELLANEOUS PUBLICATIONS

Bibliography of Highway Planning Reports. 30 cents.
Construction of Private Driveways (No. 272MP). 10 cents.
Economic and Statistical Analysis of Highway Construction Expenditures. 15 cents.
Electrical Equipment on Movable Bridges (No. 265T). 40 cents.
Federal Legislation and Regulations Relating to Highway Construction. 40 cents.
Financing of Highways by Counties and Local Rural Governments, 1931–41. 45 cents.
Guides to Traffic Safety. 10 cents.
Highway Accidents. 10 cents.
Highway Bond Calculations. 10 cents.
Highway Bridge Location (No. 1486D). 15 cents.
Highway Capacity Manual. 65 cents.
Highway Needs of the National Defense (House Document No. 249). 50 cents.
Highway Practice in the United States of America. 50 cents.
Highway Statistics, 1945. 35 cents.
Highway Statistics, 1946. 50 cents.
Highway Statistics, 1947. 45 cents.
Highway Statistics, 1948. 65 cents.
Highway Statistics, Summary to 1945. 40 cents.
Highways of History. 25 cents.
Identification of Rock Types. 10 cents.
Interregional Highways (House Document No. 379). 75 cents.
Legal Aspects of Controlling Highway Access. 15 cents.
Manual on Uniform Traffic Control Devices for Streets and Highways. 50 cents.
Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft. $1.75.
Public Control of Highway Access and Roadside Development. 35 cents.
Public Land Acquisition for Highway Purposes. 10 cents.
Roadside Improvement (No. 191MP). 10 cents.
Specifications for Construction of Roads and Bridges in National Forests and National Parks (FP–41). $1.50.
Taxation of Motor Vehicles in 1932. 35 cents.
The Local Rural Road Problem. 20 cents.
Tire Wear and Tire Failures on Various Road Surfaces. 10 cents.
Transition Curves for Highways. $1.25.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

ANNUAL REPORTS

(See also adjacent column)

Public Roads Administration Annual Reports:
1943. 1944. 1945.

MISCELLANEOUS PUBLICATIONS

Bibliography on Automobile Parking in the United States.
Bibliography on Highway Lighting.
Bibliography on Highway Safety.
Bibliography on Land Acquisition for Public Roads.
Bibliography on Roadside Control.
Express Highways in the United States: a Bibliography.
Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF DECEMBER 31, 1950

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$19,494	$11,001	$5,693	214.6	$5,288	$2,473	105.1	$13,298	$6,387	338.5	$29,587	$14,553	658.2
Arizona	5,181	1,246	892	30.1	356	248	10.9	6,738	4,810	112.2	8,340	5,950	153.2
Arkansas	7,401	8,421	4,662	222.2	2,821	1,579	52.0	16,305	8,011	422.6	27,547	14,252	696.8
California	12,688	44,480	16,724	233.6	2,588	1,343	25.0	49,344	23,963	268.7	96,412	42,030	527.3
Colorado	7,145	2,136	1,148	60.2	2,548	1,389	54.9	10,339	5,729	230.1	15,023	8,266	345.2
Connecticut	4,533	7,240	3,760	21.3	3,669	1,668	2.9	5,645	3,108	9.8	16,554	8,536	34.0
Delaware	2,212	998	600	22.3	1,019	508	5.0	5,921	2,858	35.1	7,938	3,966	62.4
Florida	8,299	12,367	6,400	232.1	10,428	5,074	242.7	12,898	6,349	279.8	35,693	17,823	754.6
Georgia	9,380	13,998	7,177	272.2	9,462	3,857	98.6	31,137	15,391	721.5	54,597	26,425	1,092.3
Idaho	7,475	7,032	4,435	236.7	2,477	1,534	100.7	5,645	2,896	125.0	15,154	8,865	462.4
Illinois	34,462	37,275	20,450	314.6	15,829	8,087	113.6	46,738	23,027	325.2	99,842	51,564	753.4
Indiana	14,960	32,447	16,740	158.4	6,932	3,346	35.5	16,050	8,291	88.7	55,429	28,377	282.6
Iowa	6,947	14,108	6,150	360.0	4,494	2,196	110.5	11,754	5,906	314.1	30,356	14,252	784.6
Kansas	10,942	10,217	4,259	921.9	1,976	991	271.9	10,523	5,264	517.6	22,716	10,514	1,711.4
Kentucky	8,456	11,082	5,351	96.8	4,826	2,419	76.5	16,133	7,954	282.4	32,041	15,724	455.7
Louisiana	8,722	15,339	7,251	112.3	6,942	3,228	56.7	20,379	10,782	222.7	42,660	21,261	391.7
Maine	4,311	5,325	2,863	65.6	1,784	898	10.3	6,546	3,480	69.8	13,655	7,241	145.7
Maryland	5,323	4,630	2,293	21.9	4,393	1,760	16.3	10,586	4,823	36.2	19,609	8,876	74.4
Massachusetts	9,259	3,150	776	.7	4,921	2,259	1.6	64,394	31,946	64.0	72,465	34,981	66.3
Michigan	15,102	12,510	6,441	387.6	10,609	5,626	169.5	38,337	15,492	279.5	61,456	27,559	836.6
Minnesota	11,434	7,378	4,379	770.5	711	339	72.7	18,593	10,193	361.3	26,682	14,911	1,204.5
Mississippi	10,107	14,241	7,510	498.3	2,725	1,302	75.6	8,141	4,174	258.4	25,107	12,986	832.3
Missouri	14,534	30,578	16,897	880.1	5,806	2,398	132.8	27,852	13,775	377.9	64,236	33,070	1,390.8
Montana	10,218	13,150	6,701	421.2	4,516	2,650	88.2	9,677	5,803	208.9	27,343	15,154	718.3
Nebraska	10,859	14,160	7,445	490.9	5,287	2,583	111.2	9,674	5,135	274.3	29,121	15,163	876.4
Nevada	5,566	2,557	2,112	38.9	972	809	59.7	3,710	3,055	125.4	7,239	5,976	224.0
New Hampshire	3,229	3,106	1,730	26.2	477	229	4.6	3,734	1,859	30.9	7,317	3,818	61.7
New Jersey	9,059	2,672	1,336	8.8	4,135	1,887	5.5	16,790	7,843	19.2	23,597	11.066	33.5
New Mexico	5,950	2,287	1,524	76.5	4,041	2,581	119.4	7,320	4,764	168.5	13,648	8,869	364.4
New York	47,090	61,865	31,534	176.7	18,877	8,427	39.0	106,574	49,859	198.4	187,316	89,820	414.1
North Carolina	9,065	16,455	7,930	417.6	6,282	2,994	99.5	22,574	10,759	519.9	45,311	21,683	1,037.0
North Dakota	6,636	7,852	4,050	1,208.1	2,742	1,315	254.3	5,128	2,546	424.2	15,722	7,911	1,886.6
Ohio	16,490	22,425	10,548	266.7	26,120	14,724	194.4	61,688	29,865	245.8	110,233	55,137	706.9
Oklahoma	9,960	7,341	4,384	68.8	10,012	5,467	122.8	19,202	9,094	421.7	36,555	18,945	613.3
Oregon	6,207	2,111	921	18.2	3,593	1,891	63.8	9,639	5,573	137.6	15,343	8,385	219.6
Pennsylvania	22,020	6,325	3,073	13.4	17,982	8,895	31.2	74,643	36,858	197.2	98,950	48,826	241.8
Rhode Island	1,956	6,743	3,371	50.9	520	260	1.4	13,224	6,710	10.4	20.487	10,341	62.7
South Carolina	6,270	9,028	4,652	212.3	2,171	1,068	73.0	7,625	4,011	196.9	18,824	9,731	482.2
South Dakota	5,802	6,903	3,911	630.1	2,153	1,251	160.0	8,534	5,265	619.0	17,590	10,427	1,409.1
Tennessee	9,203	10,889	5,259	200.2	7,054	3,419	138.0	18,281	8,260	318.2	36,224	16,938	656.4
Texas	19,812	4,491	2,295	83.9	16,453	8,466	247.1	43,279	20,110	884.2	64,223	30,871	1,215.2
Utah	4,905	3,789	2,773	82.2	763	567	25.7	4,508	3,268	140.4	9,060	6,608	248.3
Vermont	2,368	2,211	1,227	31.7	601	304	9.0	4,460	2,062	26.8	7,272	3,593	67.5
Virginia	10,203	20,951	10,374	440.2	6,289	3,109	175.5	14,182	6,927	220.4	41,422	20,410	836.1
Washington	6,864	7,195	2,625	68.5	1,560	732	22.0	21,741	10,452	141.4	30,496	13,809	231.9
West Virginia	5,131	15,088	6,174	112.4	2,795	1,406	39.6	9,444	4,775	96.7	27,327	12,355	248.7
Wisconsin	16,031	16,199	8,806	375.4	2,757	1,282	68.9	13,191	6,463	319.2	32,147	16,551	763.5
Wyoming	3,761	1,146	768	15.3	1,308	854	11.3	6,188	3,812	169.8	8,642	5,434	196.4
Hawaii	2,337	8,178	3,466	22.4	3,640	1,588	12.2	3,637	1,759	16.5	15,455	6,813	51.1
District of Columbia	3,904	4,040	2,020	4.1	703	564	.9	1,068	534	.9	5,811	3,118	5.9
Puerto Rico	3,693	13,157	6,023	66.2	755	338	2.7	9,354	4,035	37.1	23,266	10,396	106.0
TOTAL	502,986	599,513	299,883	11,761.8	267,162	134,182	4,022.2	982,365	486,065	11,911.0	1,849,040	920,130	27,695.0

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A JOURNAL OF HIGHWAY RESEARCH

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PUBLIC ROADS,
U. S. DEPARTMENT
OF COMMERCE,
WASHINGTON

Annual road costs are computed from Texas highway control section records

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Vol. 26, No. 7 *April 1951*

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BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE
E. A. STROMBERG, Editor

A Procedure for Determining the Annual Cost of a Section of Rural Highway

BY THE FINANCIAL AND ADMINISTRATIVE RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by HAROLD W. HANSEN,
Highway Engineer

IN RECENT YEARS a number of State highway departments have established highway control sections [1] as a means of coordinating records of the work performed on highways. Among the objectives involved in such undertakings is the bringing together of construction expenditures and maintenance and operation expenditures in a fashion that will establish a current and continuing record of the annual cost of each section of highway.

Such cost information, when available for the entire highway system, will provide a wealth of facts concerning the costs of providing road transportation service. Data pertaining to the annual cost of each type of surface, each type of structure, and other elements of the highway can be made available as a routine, continuing process in those States operating on a control-section basis. From such information on the cost of highways under a variety of conditions can be obtained the facts useful in short- and long-range planning and programing, and in numerous types of economic analyses and problems of economic justification.

Annual Road Cost

The term "annual road cost" as used in the following discussion refers only to the actual or estimated annual depreciation charge plus the annual expenditure for maintenance and operation. In accord with present-day accounting practice these two cost elements would be considered essential to any method of computing annual road costs.

The depreciation charge is a measure of the annual capital (construction) cost. The expenditure for maintenance and operation indicates what has been spent to preserve the highway as nearly as possible in its original condition as constructed or as subsequently improved, and for the operation of highway facilities and services to provide satisfactory and safe highway transportation.

Other elements of highway cost, such as interest, taxes, and the like, may be included in the term annual road cost depending on the use to be made of the cost information.[2] These items have been omitted from this discussion but could, if needed, be added to the costs obtained as described here.

The procedures outlined in this report have been discussed with members of State highway planning surveys, engineers, accountants, and highway officials in more than 20 States. This material has also been discussed by members of the Highway Research Board Committee on Highway Costs [3] at the twenty-eighth and twenty-ninth annual meetings of the Highway Research Board (1948 and 1949). The suggestions and advice received as a result of these discussions are gratefully acknowledged. Special recognition is due the Texas Highway Planning Survey for its cooperation in furnishing the basic data which were used to prepare the road cost charts exhibited in this report.

Depreciation

There are many mathematical formulas which can be utilized in computing depreciation charges. Some of the better known include the straight-line, production, compound-interest, sinking-fund, annuity, and fixed-percentage-of-declining-balance methods. There are others which are not as well known. In spite of the diversity of formulas the straight-line method is in almost universal use throughout the United States.[4] The principles of this method have been adapted to the determination of annual road costs.

Following its initial construction, every highway generates a demand upon current revenue for maintenance and operation and upon future revenue for rebuilding and modernization. In order that this demand may be met in a plan that will provide maximum service at minimum cost, it is essential to have knowledge of annual road costs. Such cost information is invaluable for planning and programing and, when correlated with traffic volumes, weights, surface types, and similar factors, is useful in a variety of economic analyses.

For such purposes the funds which have been expended (invested) for the construction of highways are subdivided into various fixed-asset accounts. There are eight such fixed-asset accounts recommended by the Subcommittee on Uniform Accounting of the American Association of State Highway Officials: (1) Right-of-way, (2) roadway and drainage grading and earthwork, (3) drainage structures and roadway earthwork protective structures, (4) roadway surface and base (by roadway surface type), (5) improved shoulders and approach surfacing, (6) bridges, viaducts, grade-separation structures, and tunnels (by individual structure), (7) traffic and pedestrian services, and (8) roadside development.

For purposes of an annual cost computation, the basic information required for each control section includes date of construction, cost of construction (by fixed-asset accounts), amount depleted (or, conversely, the amount salvaged) at the time of each subsequent reconstruction, age at the time of reconstruction, and other facts. As used here, depletion refers to that portion of the construction investment which is lost at the time of reconstruction, aban-

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[1] Report of the Highway Research Board Committee on Highway Costs, Memorandum No. 1. Highway Research Correlation Service Circular No. 61, May 1949.

[2] For a discussion of this subject, see *Costs of highways to the public and uses of cost computations,* Report of the Highway Research Board Committee on Highway Costs: Proceedings of the 24th annual meeting of the Highway Research Board, 1944; page 1.

[3] The members of this committee are Fred B. Farrell, *chairman*, Chief, Highway Cost Section, Bureau of Public Roads; James A. Foster, Highways and Municipal Bureau, Portland Cement Association; Carl E. Fritts, Director, Highways Division, Automotive Safety Foundation; Raleigh W. Gamble, Superintendent, Bureau of Street Construction and Repairs, Milwaukee, Wisc.; O. L. Kipp, Chief Engineer, Minnesota Department of Highways; Bertram H. Lindman, Transportation Consultant, Washington, D. C.; H. R. Wilson, Fiscal Manager, Finance and Management Division, Bureau of Public Roads; and Robley Winfrey, Research Professor of Civil Engineering, Iowa State College.

[4] *Report of Committee on Depreciation,* National Association of Railroad and Utilities Commissioners, 1943, page 89. Published by the Association, Washington, D. C.

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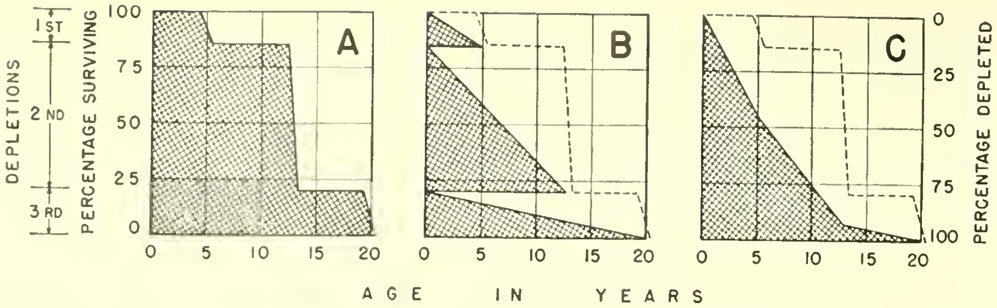

Figure 1.—A simple survivor curve, showing the age of the original investment at the time of each reconstruction and the amount of capital depleted, and the steps involved in deriving the combined straight-line depreciation curve.

donment, or transfer of the highway to another authority.[5]

This basic information can be used to construct for each fixed asset the type of graph shown in figure 1A, which was made by plotting the percentage of the original investment surviving at ½, 1½, 2½, etc., years of age and then connecting these points with straight lines, a standard practice in mortality studies of highways. The resulting curve shows the survivor history of the capital invested in a given year for the construction of one of the fixed assets (roadway and drainage grading and earthwork, drainage structures and roadway earthwork protective structures, each roadway surface and base type, etc.) comprising the highway. Such graphs can be prepared for the capital invested in each of the other fixed assets, and for additional capital invested when the highway is subsequently reconstructed. The total annual depreciation charge for the control section is built up from data computed from these individual investment survivor curves.

Depreciation Rate

To determine the rate of depreciation for a fixed asset having the survivor history indicated in figure 1A, each depletion is treated as a separate unit of property and the rate of depreciation for each depletion is found. Then the several rates of depreciation are added together to give the total rate of depreciation for that fixed asset. This is known as the unit summation method[6] for applying the principle of straight-line depreciation to a property group consisting of units having different service lives.

Figure 1B illustrates the first step in this procedure, that of determining the rate of depreciation for the individual depletions. The first reconstruction, 5 years after the initial construction, caused a depletion of 15 percent of the fixed asset. Accordingly, 15 percent of the fixed asset must be depreciated over a period of 5 years, which is the equivalent of an annual rate of 3 percent. Similarly, for the second depletion 65 percent is depreciated over a 13-year period, or an annual rate of 5 percent, and for the third depletion 20 percent is depreciated over 20 years, or 1 percent per year.

The manner by which the several rates of depreciation are combined to yield the resultant rate for the fixed asset is illustrated in figure 1C. During the period from age zero to age 5 years, the three separate rates of depreciation (3, 5, and 1 percent) are added together to obtain a total rate of depreciation of 9 percent per year. For this 5-year period, then, the total depreciation was $5\times9=45$ percent. From age 5 years to age 13 years two separate rates (5 and 1 percent) are added together to obtain the total rate of depreciation of 6 percent per year. For this 8-year period the total was $8\times6=48$ percent. In the final period, from age 13 years to age 20 years, the total rate of depreciation for the fixed asset was 1 percent per year. For this 7-year period, the total was $7\times1=7$ percent. The grand total depreciation for the three periods (45, 48, and 7 percent) is 100 percent of the original amount.

The unit summation method has the merit of relating depreciation charges to the service given by each of the units (depletions) comprising the group. It tends to reduce the underaccrual of depreciation charges on short-lived units and the overaccrual on long-lived units which is common to certain other methods for computing depreciation of group properties.

The example shown in figure 1 illustrates a situation in which all of the original fixed asset has been depleted. Comparable situations usually do not exist on highway control sections since much of the original investment (roadway and drainage grading and earthwork, for example) has not been depleted. For this undepleted capital it is necessary that the remaining life expectancy be estimated. Indications of service life may be obtained from the analysis of construction investment retirements, a phase of the road life studies being conducted by the State highway planning surveys in cooperation with the Bureau of Public Roads.

The share of construction costs which may properly be charged to a control section during a given year is found by adding together the depreciation charges which have been computed for that year for all improvements built within the control section. These figures in turn are made up of actual or estimated depreciation charges for the individual fixed-asset accounts comprising the improvement in accordance with the methods outlined above.

Forms for Computations

Depreciation computations can be greatly facilitated by the use of well-designed forms. A form which provides for all possible adjustments to depreciation rates and for all future additions of data may become unnecessarily complicated. A simpler form will usually prove more desirable even though it may require the occasional preparation of a new depreciation sheet for the control section when an appreciable revision is caused by later reconstruction.

Forms 1 and 2, shown in figure 2, are examples of the type of form used for computing and summarizing depreciation data for control sections and are shown in this report for illustrative purposes only. To facilitate comparing the cost of one section of road with the cost of any other, these particular forms provide for showing depreciation as an annual charge on a per-mile basis for any selected segment within the control section and for the control section as a whole. It will be noted also that these forms provide for adjusting depreciation charges to a common price level in order that all costs may be expressed in terms of dollars having approximately the same purchasing power.

Correlating maintenance and operation expense with depreciation charges can be readily undertaken in those States that have established control sections since this is one of the basic functions of the control section procedure. In the same way that depreciation charges are adjusted to a common price level, maintenance and operation expense is adjusted by means of a maintenance price index to a common price level. The same base periods should be used for both the construction and maintenance price indexes.

Form 3 (fig. 2) is an example of a form for summarizing depreciation charges and maintenance and operation expense on a per-mile basis for a given control section. This particular form is for illustrative purposes only and should be designed in each instance to fit the needs in a given State. For example, bridge data are not recorded on form 3, but may readily be included, if desired.

The annual road cost per mile for the control section is obtained by adding the total annual depreciation charge per mile for the control section and the corresponding maintenance and operation expense per mile.

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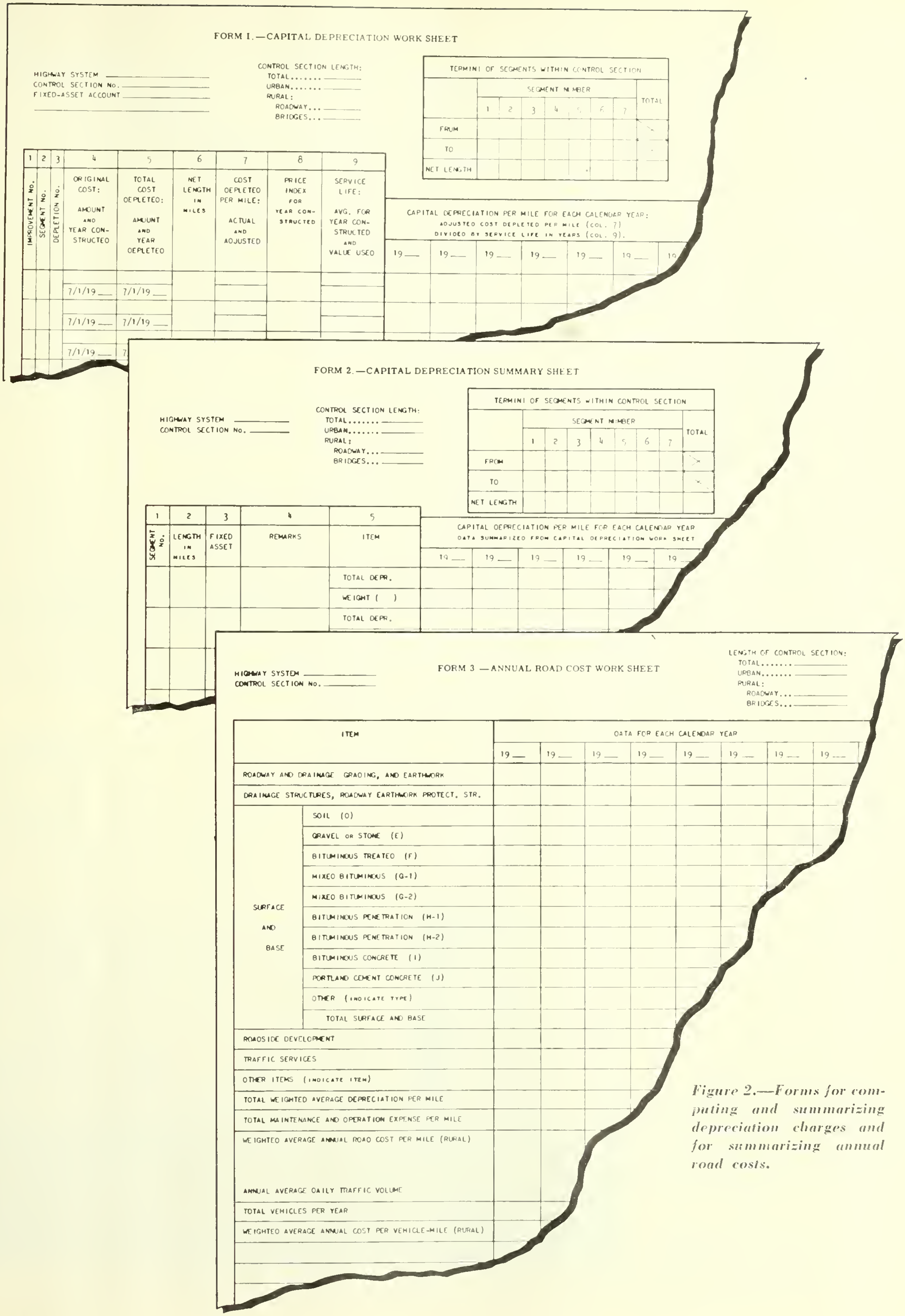
[5] Methods of developing a progressive record of construction investment are outlined in the Bureau of Public Roads Planning Survey Memorandum No. 322, June 30, 1939: *Further suggestions on the summary and tabulation of the road life studies.*

[6] *Depreciation of Group Properties*, by Robley Winfrey, Iowa Engineering Experiment Station Bulletin 155, 1942; Iowa State College, Ames, Iowa.



FORM 1.—CAPITAL DEPRECIATION WORK SHEET

HIGHWAY SYSTEM ______
CONTROL SECTION No. ______
FIXED-ASSET ACCOUNT ______
______

CONTROL SECTION LENGTH:
TOTAL....... ______
URBAN....... ______
RURAL:
ROADWAY... ______
BRIDGES... ______

TERMINI OF SEGMENTS WITHIN CONTROL SECTION	SEGMENT NUMBER							TOTAL
	1	2	3	4	5	6	7	
FROM								
TO								
NET LENGTH								

1	2	3	4	5	6	7	8	9	CAPITAL DEPRECIATION PER MILE FOR EACH CALENDAR YEAR: ADJUSTED COST DEPLETED PER MILE (COL. 7) DIVIDED BY SERVICE LIFE IN YEARS (COL. 9).						
IMPROVEMENT No.	SEGMENT No.	DEPLETION No.	ORIGINAL COST: AMOUNT AND YEAR CONSTRUCTED	TOTAL COST DEPLETED: AMOUNT AND YEAR DEPLETED	NET LENGTH IN MILES	COST DEPLETED PER MILE: ACTUAL AND ADJUSTED	PRICE INDEX FOR YEAR CONSTRUCTED	SERVICE LIFE: AVG. FOR YEAR CONSTRUCTED AND VALUE USED	19__	19__	19__	19__	19__	19__	19
			7/1/19__	7/1/19__											
			7/1/19__	7/1/19__											
			7/1/19__	7											

FORM 2.—CAPITAL DEPRECIATION SUMMARY SHEET

HIGHWAY SYSTEM ______
CONTROL SECTION No. ______

CONTROL SECTION LENGTH:
TOTAL....... ______
URBAN....... ______
RURAL:
ROADWAY... ______
BRIDGES... ______

TERMINI OF SEGMENTS WITHIN CONTROL SECTION	SEGMENT NUMBER							TOTAL
	1	2	3	4	5	6	7	
FROM								
TO								
NET LENGTH								

1	2	3	4	5	CAPITAL DEPRECIATION PER MILE FOR EACH CALENDAR YEAR DATA SUMMARIZED FROM CAPITAL DEPRECIATION WORK SHEET					
SEGMENT No.	LENGTH IN MILES	FIXED ASSET	REMARKS	ITEM	19__	19__	19__	19__	19__	19
				TOTAL DEPR.						
				WEIGHT ( )						
				TOTAL DEPR.						

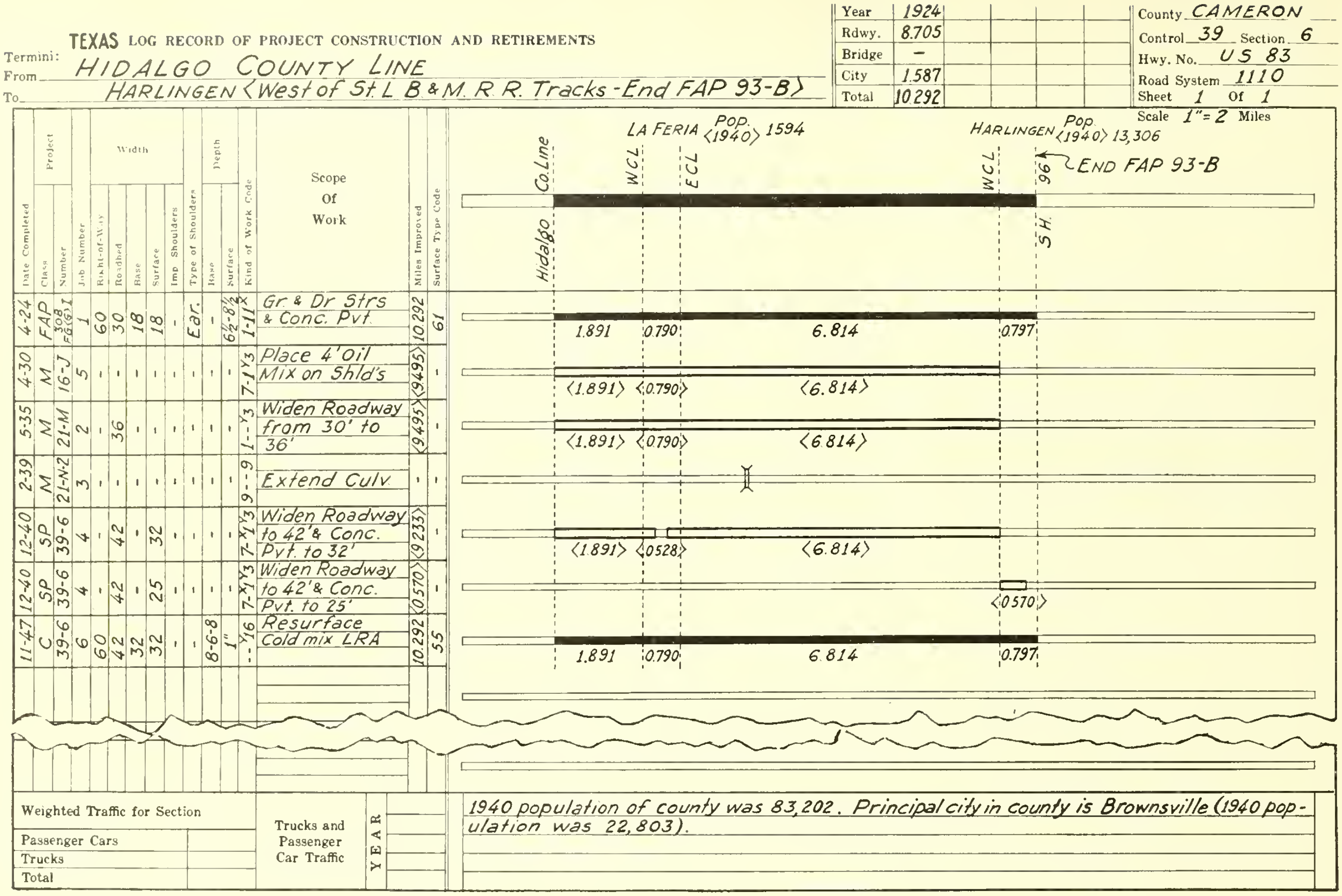
FORM 3 —ANNUAL ROAD COST WORK SHEET

HIGHWAY SYSTEM ______
CONTROL SECTION No. ______

LENGTH OF CONTROL SECTION:
TOTAL....... ______
URBAN....... ______
RURAL:
ROADWAY... ______
BRIDGES... ______

ITEM		DATA FOR EACH CALENDAR YEAR							
		19__	19__	19__	19__	19__	19__	19__	19__
ROADWAY AND DRAINAGE GRADING, AND EARTHWORK									
DRAINAGE STRUCTURES, ROADWAY EARTHWORK PROTECT. STR.									
SURFACE AND BASE	SOIL (D)								
	GRAVEL OR STONE (E)								
	BITUMINOUS TREATED (F)								
	MIXED BITUMINOUS (G-1)								
	MIXED BITUMINOUS (G-2)								
	BITUMINOUS PENETRATION (H-1)								
	BITUMINOUS PENETRATION (H-2)								
	BITUMINOUS CONCRETE (I)								
	PORTLAND CEMENT CONCRETE (J)								
	OTHER (INDICATE TYPE)								
	TOTAL SURFACE AND BASE								
ROADSIDE DEVELOPMENT									
TRAFFIC SERVICES									
OTHER ITEMS (INDICATE ITEM)									
TOTAL WEIGHTED AVERAGE DEPRECIATION PER MILE									
TOTAL MAINTENANCE AND OPERATION EXPENSE PER MILE									
WEIGHTED AVERAGE ANNUAL ROAD COST PER MILE (RURAL)									
ANNUAL AVERAGE DAILY TRAFFIC VOLUME									
TOTAL VEHICLES PER YEAR									
WEIGHTED AVERAGE ANNUAL COST PER VEHICLE-MILE (RURAL)									

Figure 2.—Forms for computing and summarizing depreciation charges and for summarizing annual road costs.



TEXAS LOG RECORD OF PROJECT CONSTRUCTION AND RETIREMENTS

Termini:
From *HIDALGO COUNTY LINE*
To *HARLINGEN <West of St. L. B & M. R. R. Tracks - End FAP 93-B>*

Year	1924				
Rdwy.	8.705				
Bridge	–				
City	1.587				
Total	10.292				

County *CAMERON*
Control *39* Section *6*
Hwy. No. *US 83*
Road System *1110*
Sheet *1* Of *1*
Scale *1" = 2* Miles

Date Completed	Project		Job Number	Width				Imp Shoulders	Type of Shoulders	Depth		Kind of Work Code	Scope Of Work	Miles Improved	Surface Type Code
	Class	Number		Right-of-Way	Roadbed	Base	Surface			Base	Surface				
4-24	FAP	308 FG(G) I	1	60	30	18	18	-	Ear.	-	6½-8½	1-11X	Gr. & Dr Strs & Conc. Pvt.	10.292	61
4-30	M	16-J	5	-	-	-	-	-	-	-	-	7-1Y3	Place 4' Oil Mix on Shld's	<9495>	-
5-35	M	21-M	2	-	36	-	-	-	-	-	-	1--Y3	Widen Roadway from 30' to 36'	<9495>	-
2-39	M	21-N-2	3	-	-	-	-	-	-	-	-	9--9	Extend Culv.	-	-
12-40	SP	39-6	4	-	42	-	32	-	-	-	-	7-X1Y3	Widen Roadway to 42' & Conc. Pvt. to 32'	<9233>	-
12-40	SP	39-6	4	-	42	-	25	-	-	-	-	7-X1Y3	Widen Roadway to 42' & Conc. Pvt. to 25'	<0570>	-
11-47	C	39-6	6	60	42	32	32	-	-	8-6-8	1"	--Y16	Resurface Cold mix LRA	10.292	55

Weighted Traffic for Section		Trucks and Passenger Car Traffic	YEAR
Passenger Cars			
Trucks			
Total			

1940 population of county was 83,202. Principal city in county is Brownsville (1940 population was 22,803).

Figure 3.—Log record of project construction and retirements for Texas control section 39-6.

At the bottom of form 3, space should be provided for special computations. One such possible computation is the cost per vehicle-mile, as shown on the form. Others may be entered on the form at the discretion of the highway department to show trends in the annual costs of the control section in relation to other factors.

Annual Road Cost Data

As an example of annual cost determinations, a highway in Texas is here considered. Figure 3 shows one of the construction project log records prepared by the Texas State Highway Department for control section 39–6 on a portion of U S 83 in Cameron County, the southernmost county in the State. This control section begins at the west county line and extends eastward to the intersection with State Route 96 in the city of Harlingen. The total length of the control section is 10.292 miles, but only the rural portion of 8.705 miles is embraced in the following discussion.

A view of the roadway looking to the west from just inside the city limits of Harlingen is shown in figure 4, and another part of the control section is depicted on the cover. The existing three-lane surface is 32 feet wide and consists of a 1-inch limestone rock-asphalt mat on an old portland cement concrete base.

Although agriculture plays a dominant role in the economy of the county, one-half of the population lives in urban areas. Interurban bus service is scheduled at frequent intervals along this route. There are a number of manufacturing establishments in the county, some of which depend on local crops for raw materials. Many of these establishments rely upon motor carriers to support their operations. The area also has a modest tourist trade. Average traffic on control section 39–6 was 3,389 vehicles per day in 1936. Following a wartime decline

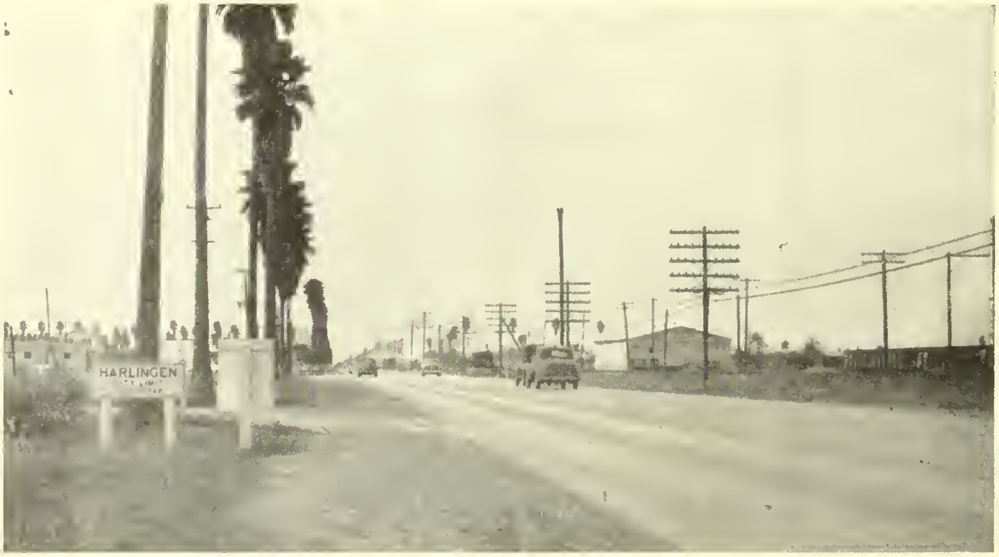


Figure 4.—View of the eastern end of Texas control section 39-6.

to 2,659 in 1943, the average traffic increased to 5,542 vehicles per day in 1948.

Annual roadway costs for an average mile of control section 39–6 are shown in the upper left portion of figure 5. In this illustration all costs have been adjusted to a 1937–41 price level. Bridge costs were omitted from the computations of costs, but could be included, if desired, with no change in the general procedure.

The annual depreciation charges, as obtained from the investment study, extend back to 1924. Data on maintenance and operation expense, however, date back only to 1936. The State's control-section setup was not in full operation before that date, and maintenance and operation expense could not be correlated with construction costs for the years prior to 1936. Consequently, the data presented in figure 5 begin with the year 1936.

Only the total annual depreciation and the total maintenance and operation expense are shown in figure 5. Similar charts could be prepared for each of the fixed-asset accounts, if desired, by selecting the appropriate data from the annual road cost work sheet for the control section.

For purposes of general illustration, annual average daily traffic information has been obtained for control section 39–6, and an annual cost per vehicle-mile computed. These data are also shown in the left side of figure 5. The true significance of such a trend for a particular control section can, of course, be properly appraised only after consideration and study of many factors, such as composition and type of traffic, frequency of heavy axle loads, subgrade conditions, degree of maintenance afforded, and type and width of surface and shoulders.

Cost Trends

In order to smooth out the somewhat erratic trends characteristic of these data when presented on an annual basis, averages for a group of years may prove more useful. The right side of figure 5 shows the annual cost data for control section 39–6 on the basis of 5-year averages. In this case the 5-year averages are plotted at the last year of each 5-year group. Thus, the average for 1936–40 is plotted at 1940, and so on. The trends are more apparent when the data are arranged this way, thus making them more acceptable for general administrative use. The average annual daily traffic and annual cost per vehicle-mile, on a 5-year average basis, are also shown in the right side of figure 5.

Computations similar to the foregoing have been made for other control sections in Texas and other States. The annual cost per mile, traffic volume, and annual cost per vehicle-mile for four additional control sections in Texas are shown in the upper

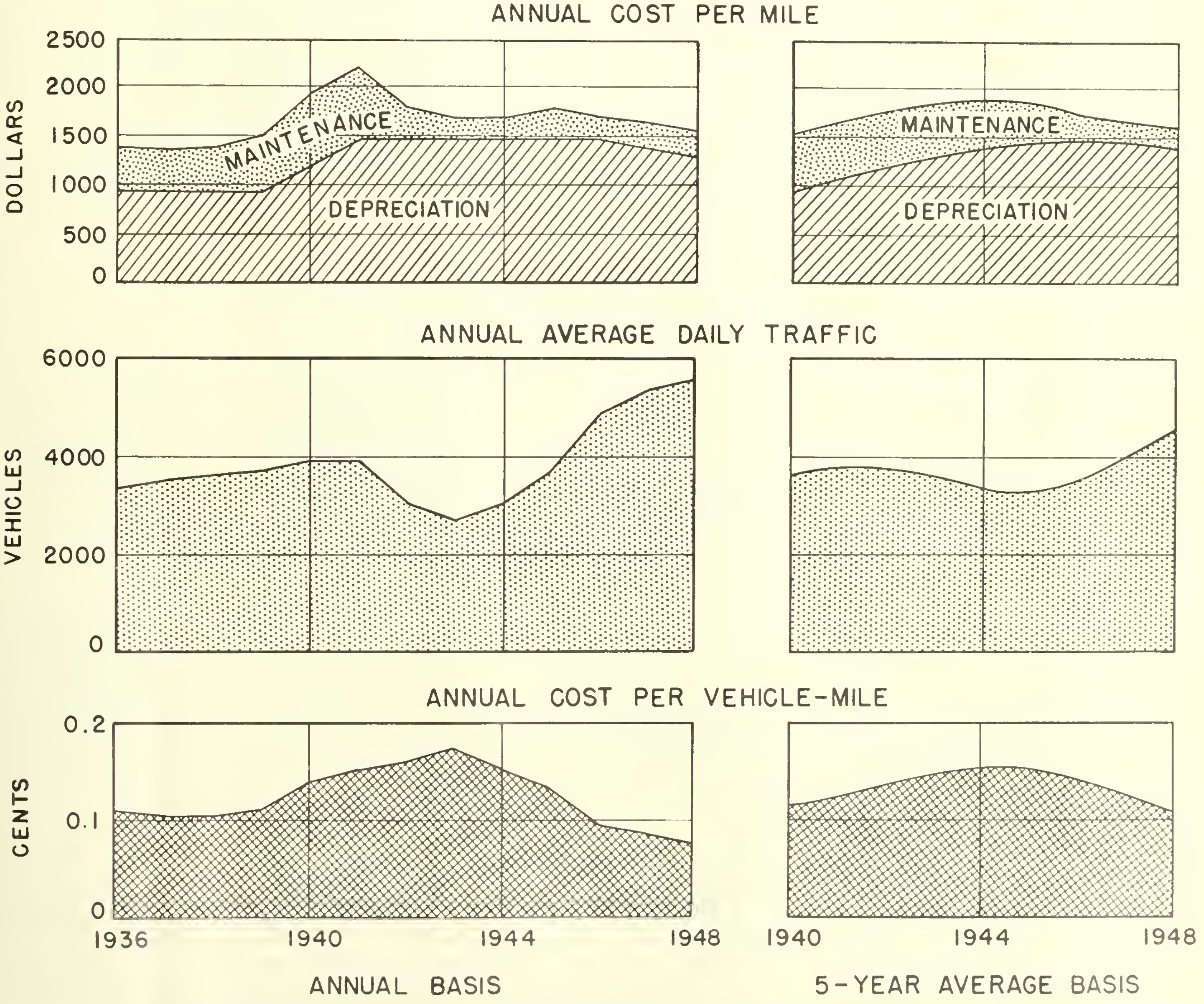


Figure 5.—Annual cost per mile, annual average daily traffic, and annual cost per vehicle-mile, on an annual basis and a 5-year average basis, for Texas control section 39-6 (data adjusted to 1937–41 prices).

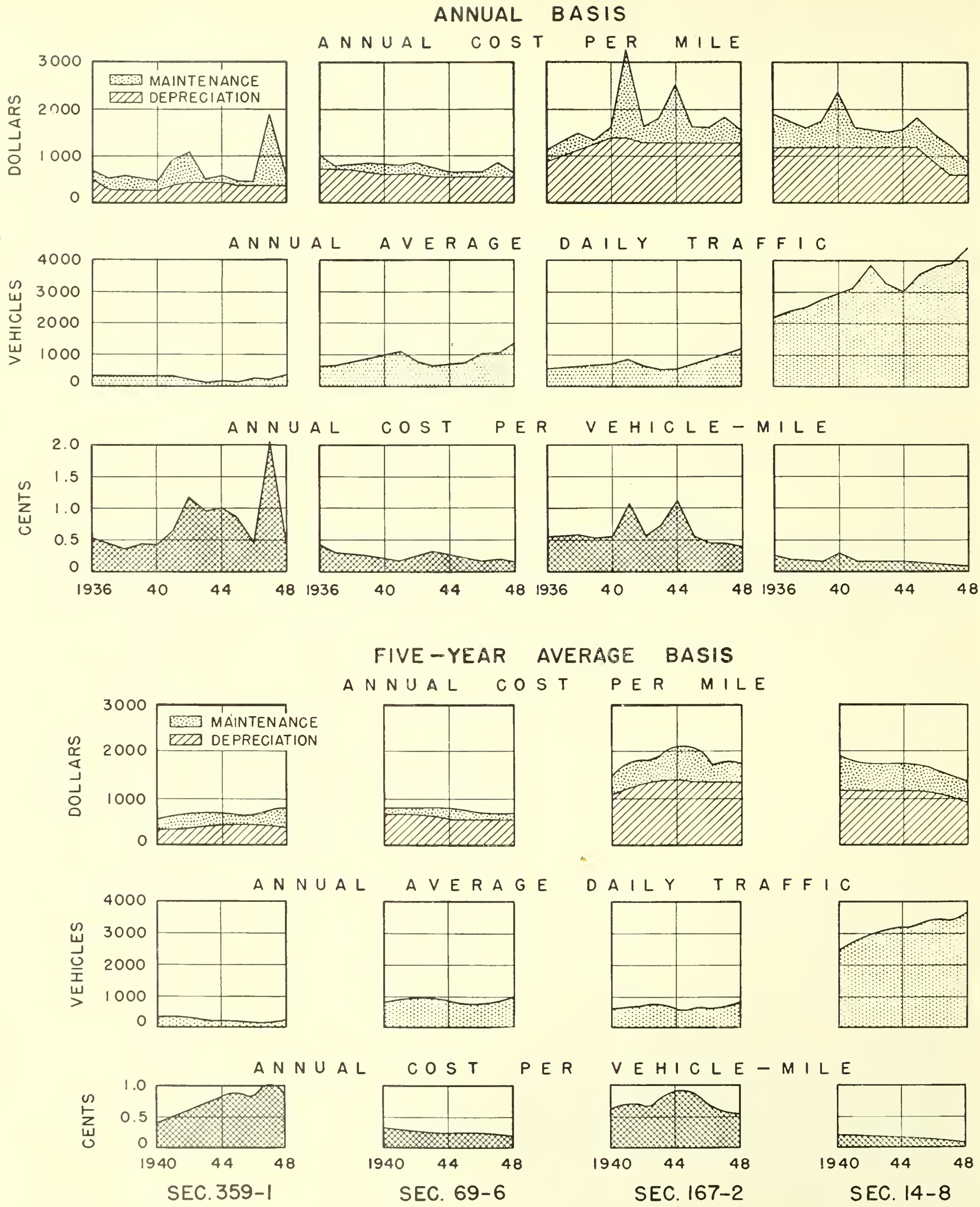


Figure 6.—Annual cost per mile, annual average daily traffic, and annual cost per vehicle-mile, on an annual basis and a 5-year-average basis, for four Texas control sections (data adjusted to 1937–41 prices).

part of figure 6. Similarly, the 5-year-average annual cost per mile, 5-year-average daily traffic, and 5-year-average annual cost per vehicle-mile for the same sections are shown in the lower part of figure 6. These control sections, which were selected at random, are briefly described in table 1. The cost data shown here are not necessarily representative of the average cost of these road types and consequently conclusions should not be drawn from this small sample as to relative cost.

Table 1.—*Length, type, and location of control sections*

Control section number	Length	Number of lanes	Surface type	Route	County	Location
	Miles					
359-1	7.102	2	Bituminous surface treated.	Texas 119	DeWitt	From Yorktown southeast toward Goliad.
69-6	9.768	2	Bituminous penetration.	U S 87	Tom Green	From Coke County line southeast toward San Angelo.
167-2	15.246	[1] 2	Bituminous concrete	U S 54	El Paso	From El Paso north to New Mexico State line.
14-8	10.062	[2] 2	Portland cement concrete.	U S 77 and 81	McLennan	From Hill County line south toward Waco.

[1] 4.776 miles adjacent to El Paso were widened to four lanes in 1941.
[2] 0.544 mile at the south end was built (1934) as three lanes.

New Publications

THE MATHEMATICAL THEORY OF VIBRATION IN SUSPENSION BRIDGES

The Bureau of Public Roads has recently published *The Mathematical Theory of Vibration in Suspension Bridges*, a 450-page report of a study undertaken under the auspices of the Advisory Board on the Investigation of Suspension Bridges. The book is for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at $1.25 a copy.

The Advisory Board, established subsequent to the collapse of the Tacoma Narrows Bridge in 1940, has sponsored a series of investigations seeking to determine the causes of suspension bridge vibration, to develop a rational theory explanatory of the phenomenon, and to work out methods of design practice which will reduce these dangerous motions.

The Mathematical Theory of Vibration in Suspension Bridges develops mathematical analyses of the dynamic problem and suggests experimental data needed to support them. Authors of this highly technical study are Dr. Friedrich Bleich, Consulting Engineer, American Institute of Steel Construction; Dr. C. B. McCullough, Assistant Chief Engineer, Oregon State Highway Commission; Richard Rosecrans, Structural Research Engineer, Oregon State Highway Commission; and George S. Vincent, Principal Highway Bridge Engineer, Bureau of Public Roads. Unfortunately, both Dr. Bleich and Dr. McCullough died before their work appeared in published form.

The content of the publication is indicated by the subjects of the major sections:

Chapter 1.—History of troublesome dynamic stress effects in suspension bridges; a general statement of the dynamic problems; the mathematical theory of vibration problems.

Chapter 2.—A general statement of the problem and determination of physical data needed for its solution.

Chapter 3.—Frequencies, modes of vibration, and energy storage in freely vibrating suspension bridges with hinged stiffening frames.

Chapter 4.—Application of the energy method to the analysis of the frequencies, modes of vibration, and energy storage capacity in freely vibrating suspension bridges with continuous stiffening frames.

Chapter 5.—Influence of various design factors on frequencies, modes of motion, and energy storage.

Chapter 6.—Consideration of structural damping in suspension bridges.

Chapter 7.—The flutter theory of truss-stiffened suspension bridges under wind action.

Chapter 8.—The linearized deflection theory developed for suspension bridges having tower stays.

Chapter 9.—Experimental verification of the linearized deflection theory.

SELECTED BIBLIOGRAPHY ON HIGHWAY FINANCE

The Bureau of Public Roads has recently published a *Selected Bibliography on Highway Finance* which provides nearly 1,400 annotated references, for the years 1939–49, in the field of highway taxation and finance in the United States. Emphasis is placed chiefly on theory, but basic sources of pertinent statistics are included. The bibliography may be purchased from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 55 cents a copy.

The search for solutions to the problems currently encountered by all levels of government in obtaining adequate revenues for highway operation and capital outlay has intensified the value of all types of information on finance. The *Selected Bibliography on Highway Finance* is the complete key to this need.

The references in the bibliography are arranged according to a useful classification scheme under seven major topics—general discussions, taxation, expenditures, borrowing, financial programs and plans, miscellaneous topics, and statistics. Both geographical and author indexes are included in the publication.

Brief annotations accompany most entries to indicate the general nature of content, but evaluation of importance, accuracy, or validity has not been attempted. The references were carefully selected, however, so that all included are of some significance.

While the bibliography covers intensively the years 1939–49, a limited number of significant references from earlier years are also included, as are available items published in the first four months of 1950.

HIGHWAY STATISTICS, 1949

Now available is the Bureau of Public Roads' *Highway Statistics, 1949*, the fifth of the bulletin series presenting annual statistical and analytical tables of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, financing of State highways, and highway mileage. Included for the first time in the annual publication is information concerning the financing of highways by county and local rural governments.

Highway Statistics, 1949 is for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 55 cents a copy. The full series of the annual bulletins are available from the Superintendent of Documents, as indicated on the inside back cover of PUBLIC ROADS.

Methods for the Determination of Soft Pieces in Aggregate

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by D. O. WOOLF, Senior Materials Engineer

The classification of materials in aggregates as "soft pieces" is subject to wide variation and loose definition. Materials classed as soft pieces in various State specifications also include those which more properly should be defined as unsound, light-weight, brittle, friable, highly absorbent, or combinations of these. As pointed out in this article, specifications for aggregates should define and limit soft pieces as such, and the presence of other deleterious substances should be separately restricted.

The study reported here included an examination of existing methods, and an attempt to develop new ones, for the determination of the presence and proportion of soft pieces in aggregate. All commonly known test methods, and many specially devised ones, were studied. It was concluded that the only method suitable for both field and laboratory use is the scratch hardness test using a hard, yellow brass scribe. For convenience, a wooden pencil with a brass core is suggested.

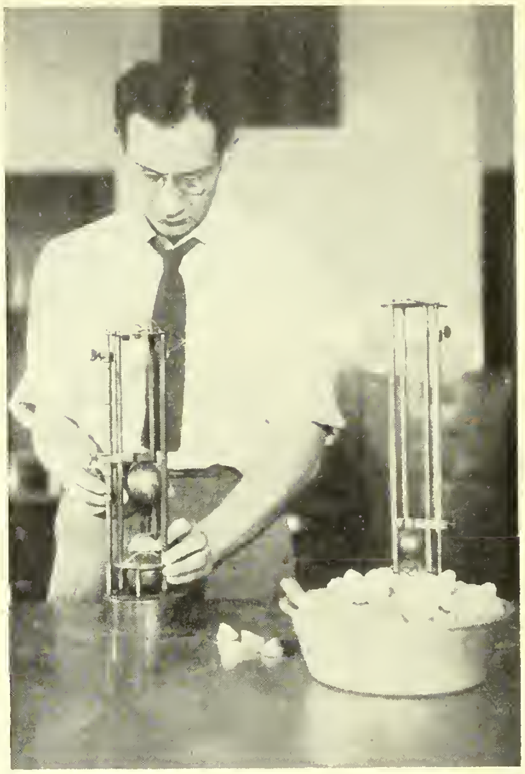


Figure 1.—Toughness test for gravel, using the 2½-inch ball.

IN 1941 the Bureau of Public Roads undertook an investigation of methods of testing coarse aggregates to determine the content of soft pieces, at the request of Subcommittee IX of Committee C-9 of the American Society for Testing Materials. It was hoped that the results of this investigation would direct attention toward the development of a new method of testing aggregates for soft pieces, or identify existing methods which would be suitable for this purpose.

This paper describes the test methods employed and gives typical test values which were obtained. With each description of a method of test are presented the conclusions obtained from the test data: Whether the method appears to be useful, and the difficulties encountered in its performance. Throughout the investigation, efforts were made to develop a method which would be suitable for use in the field. It is thought that this determination—the detection of the amount of soft pieces in aggregates—is properly a duty of the inspector in the field. Any method which the inspector can use can also be performed in the laboratory; consequently, the program of the investigation was planned with consideration of field use as a primary requisite. In some of the methods employed, apparatus suitable only for use in the laboratory was used. In most of these cases, however, plans were prepared for the construction of equipment sufficiently portable to permit its installation at aggregate plants.

Conclusions

The tests reported were made in an attempt to develop a rational method of testing aggregates for soft-piece content. Although most tests used standard or readily procurable apparatus, some rather fanciful or weird contraptions were designed or actually constructed for this purpose. On the whole, a thorough study has been made of all different methods of test which could be expected to give information of value.

Of all methods tried, the only one considered suitable for both laboratory and field use is the scratch hardness test using a hard, yellow brass scribe. For convenience, a pencil with a brass core is suggested.

Attention is called to the rather frequent practice, in writing specifications for aggregates, of using the soft-piece classification as a catch-all for many different types of deleterious substances. If it is desired to limit types of deleterious substances other than soft pieces, these other types should be mentioned specifically and separately, in the specifications, from pieces which are merely soft.

Classification of Materials

Prior to starting this investigation of methods of test, an attempt was made to determine the character of the materials which the various highway authorities classify as soft pieces. The State highway departments were requested to submit samples of such soft pieces as are encountered in each State, and to furnish information relating to their particular conditions insofar as this problem is concerned. The information requested was as follows:

1. Names of the different materials which may be classified as soft pieces.
2. Names of such materials found generally throughout the State or in large areas of the State.
3. Amount of soft pieces permitted in specifications for pavement surface courses.
4. Method of test or identification used by laboratory operators or field inspectors.
5. Names of the materials occurring naturally which are considered to be objectionable for surface courses but which are not included in the class of soft pieces.

Replies were received from practically all of the State highway departments. Many were in sufficient detail, but a few were expressed in generalities, or in terms which appear to be of local usage only. From the information received, table 1 reports a list of materials which are classed as "soft pieces," the number of States which classify these materials as soft, and the number of States which report the materials to be widely distributed.

At first glance, many of the materials listed in table 1 appear to be included in the designation of soft pieces by mistake.

For example, granite is placed in the classification of soft pieces by two States, although the word "granite" usually conveys the impression of a hard and enduring stone. A number of other apparent inconsistencies exists. However, some interpretation of these classifications is probably permissible.

There is little doubt that the inclusion of granite in a classification of soft pieces is meant to refer to material of a granitic nature which has weathered to such an extent that the component crystals are soft or poorly bonded to each other. Similar interpretations can be made of a number of the materials included in table 1 and, in general, most of the materials mentioned can be placed in one or more of the following types:

1. Soft.
2. Unsound.
3. Light weight.
4. Brittle.
5. Friable.
6. Highly absorbent.

Some materials may be both soft and friable, or soft and brittle, or different samples of the same material may be placed properly in separate groups. Consequently, without reference to actual samples, a proper grouping of materials by name alone is probably not feasible. Certain materials mentioned in table 1, such as brownstone, micaceous granite, and limonite, are not identified definitely by the names given, although use of such terms may be justified in a limited area where the persons concerned would have an understanding of their application. However, as these terms may not be widely used to describe materials of definite physical characteristics, some thought could be given toward the use of more descriptive names, or to the establishment and use of a systematized glossary of terms relating to the materials under consideration.

Materials classed as deleterious but not considered as soft pieces by the reporting authority are listed below:

Alkaline reactive
Amphibolite
Chert
Chert, opaline
Chert, unsound
Clay lumps
Coal
Coated material
Dolomite, some
Flint
Glassy rock
Gneiss, soft
Granite
Granite, soft
Granite, some
Gravel, a quartz
Hydrophilic rock
Limestone, argillaceous
Limestone, siliceous
Mica
Mud balls
Obsidian
Ochre
Pyrite in rock
Quartzite
Sandstone, hard absorbent
Sandstone, soft
Shale
Shale, hard
Shale, opaline cherty
Shale, soft
Shale, some
Shell
Slate
Sulphates–sulphides, iron, in rock
Thin or elongated pieces
Unsound pieces

Table 1.—*List of materials classed as soft pieces and number of States giving this classification or having material widely distributed*

Material	Number of States—		Material	Number of States—	
	Classifying as soft	Reporting wide distribution		Classifying as soft	Reporting wide distribution
Basalt, disintegrated	1	1	Limestone, soft	3	1
Brownstone	1	1	Limestone, some varieties	1	1
Caliche, hard	1	0	Limestone, weathered	4	3
Chert	2	1	Limonite	3	2
Chert, chalky	1	0	Marble	1	1
Clay balls	2	1	Ochre	3	2
Clay, iron bearing	1	0	Quartz, sugar	1	1
Coal	2	1	Quartz, weathered	2	2
Concretion, calcareous	1	1	Rock, disintegrated	2	1
Conglomerate	1	0	Rock, weathered	2	2
Dolomite, weathered and porous	1	1	Rottenstone	1	1
Earth, diatomaceous	1	1	Sandstone	9	9
Feldspar	1	1	Sandstone, argillaceous	1	1
Felsite	1	0	Sandstone, friable	1	1
Floaters	1	1	Sandstone, friable arkosic	1	0
Gneiss	1	0	Sandstone, soft	5	4
Gneiss, micaceous	1	1	Schist	5	3
Gneiss, weathered	1	1	Schist, some	1	1
Granite	2	1	Schist, micaceous	1	0
Granite, decomposed	1	1	Schist, talc	1	0
Granite, disintegrated	4	3	Schist, weathered	2	2
Granite, micaceous	1	1	Scoria	1	1
Granite, weathered	3	3	Scoria, certain grades of	1	1
Gravel, cemented	2	1	Shale	11	7
Gravel, magnesia	1	1	Shale, clay	1	1
Iron clay balls	1	1	Shale, diatomaceous	1	1
Iron oxide	2	1	Shale, disintegrated	1	1
Limestone, argillaceous	2	1	Shell	2	1
Limestone, porous	1	1	Soapstone	1	0
Limestone, shelly	1	0	Volcanic rock, coarse	1	1

It is interesting to note, in this list of 37 items, that 17 are placed in the category of soft pieces by other States. This illustrates one of the difficulties encountered in attempting to coordinate requirements or tests for soft pieces in aggregate. Unless there can be some agreement as to what constitutes a soft piece, work along the proposed line may not be of suitable value.

Table 2 presents a compilation of the various State specifications for limitation of soft pieces. Most specifications limit the content of soft pieces of aggregate to 5 percent or less. In a few cases, limits for specific types of soft particles are given, but usually the specifications fail to identify the materials covered by the requirement.

Test Methods Reported by States

The following methods of test to determine the content of soft pieces of aggregate were reported by the States:

	Number of States
Los Angeles abrasion test	10
Breakage test under roller	1
Visual inspection	14
Scratch test	4
Hand hammer or compression test	4
Douglass stone meter	2
Specific gravity	2
Solubility in acid	1
Sulfate soundness test	2
Absorption test	1
Flotation by heavy liquid	1
Gravel impact test	1
Deval abrasion test	1

The methods most commonly used are visual inspection, the Los Angeles abrasion test, the scratch test, and a test by use of a hand hammer or compression machine. These and the other tests listed are of two types—a test in which the individual particles are examined separately, and a test in which the effect of soft pieces on a characteristic of the entire material is determined. It should be noted that, with the exception of the requirement involving the use of the sodium-sulfate test, all of the specification requirements shown in table 2 necessitate the use of the first type of test and, in the one exception, this type of test is used in part. Consequently, there appears to be a strong opinion to the effect that determinations of the content of soft pieces should be based on actual count or weight of particles rather than through the use of an indirect method involving some characteristic pertaining to the whole sample.

A number of the States report the use of certain methods of test—the Los Angeles abrasion test for example—for determining

Table 2.—*State specification limitations for soft pieces*

Item	Specification limit	Number of State specifications
	Percent	
Soft pieces	2	2
Do	3	3
Do	5	4
Do	10	1
Do	Free from excess.	1
Soft pieces, bituminous aggregate	1	2
Soft pieces, concrete aggregate	2	2
Do	3	1
Soft pieces Concrete aggregate	6	1
Cover material	7	
Oil mix material	12	
Stabilized material	20	
Clay lumps	0.5	1
Shale	2.0	
Sp. gr. less than 1.95	2.0	
Total shale, coal, clay lumps, and soft fragments	5	1
If loss in Na_2SO_4 test is less than 2 percent, soft shale may not exceed	10	1
If loss is 2.0 percent or greater, soft shale is limited to	6	

the presence of soft pieces in aggregate, but fail to include numerical values in their reported limitation requirements for soft pieces which are applicable to these methods. Although the reporting authorities may consider the methods mentioned to be suitable for use, it is probable that lack of sufficient test data has so far prevented the establishment of specification requirements.

Eighty-four samples of aggregates were submitted by the States in response to the request for typical samples of material composed of soft pieces. In a number of cases the samples were confined, as requested, to a given type of material. However, many samples were found to be composed of a number of kinds of material differing as greatly as limestone and gneiss. It is possible that the request for samples composed wholly of soft pieces was misunderstood, and that some samples were submitted which contained a small amount of soft pieces as it naturally occurred in these materials.

From the number of varieties of materials submitted as representing soft pieces, it is apparent that there is no concordance of opinion as to the kind of material which should be thus described. It is further apparent that the term "soft pieces" is used as a catch-all description of a number of different types of possibly undesirable material in aggregates. Prior to spending much time in testing materials, it was realized that some rational conception must be formed of the type of material to be identified by the test procedure. Choice must be made between the application of the term "soft pieces" to pieces of aggregate which are actually soft, or the application of the same term to pieces of a wide variety of characteristics. Although the latter may be undesirable for use in construction, this is their only common feature. As shown in the samples submitted, these pieces may be hard or soft, tough or brittle, sound or unsound; in fact, they include the whole gamut of physical properties of aggregates. To this question of choice, there can be but one answer: If these tests are to identify soft pieces, the only problem to be considered is whether the piece under test is hard or soft. During the course of this investigation, tests foreign to a strict hard-or-soft determination were made. In these tests it was hoped that some correlation between the hardness of the particle and some other characteristic of interest could be established. As a general rule, the results were rather disappointing. Although in some cases, with selected samples, definite correlations were found between the hardness and some other characteristic of a material, when a wider variety of samples was considered the agreement between the two characteristics appeared to be largely a matter of chance.

Classification of Test Methods

The tests made in this investigation may be grouped in the following classifications:

1. Visual inspection.
2. Scratch hardness.
3. Specific gravity and absorption.
4. Resistance to impact.
5. Resistance to abrasion.
6. Resistance to static loads.
7. Soundness.

In some cases, the tests were made on the material as received; that is, without preliminary separation into individual sizes. In most tests, however, the size of the piece affected the test result to a marked extent, and it was necessary to sieve the test sample into separate sizes. The sizes generally used were 2- to 1½-inch, 1½- to 1-inch, 1- to ¾-inch, and ¾- to ½-inch. Pieces smaller than ½-inch were found to be difficult to test by some methods, and it was thought that it would be sufficient, for the present at least, to study methods of testing the larger pieces only.

In the first series of tests made on the 84 samples submitted by the State highway departments, the samples were separated into three groups with respect to hardness of the material: Soft, hard, and borderline. This was done by a combination of visual inspection and the use of a scratch test employing a steel knife. It was realized that these methods are not dependable for separating any and all materials into hard and soft classifications. However, they can be used to separate unquestionably soft material from unquestionably hard material. The materials that were found to be neither hard nor soft were classified as borderline materials and excluded from further tests.

Visual inspection of aggregates for the presence of soft pieces is associated with what might be called the luster or appearance by reflected light, and the degree of bonding. Pieces of aggregate with a glassy or stony luster are usually classed as hard; those with a dull or earthy appearance are classed as soft. Compact materials are usually placed in the hard classification, while those which have a loose or friable texture are considered as soft.

Table 3.—*Use of the absorption test for the identification of hard and soft materials*

Item		Size of pieces			
		2- to 1½-inch	1½- to 1-inch	1- to ¾-inch	¾- to ½-inch
Soft Material:					
Number of samples		4	30	31	20
Absorption:					
Minimum	percent	2.6	2.6	3.2	4.3
Maximum	do	23.4	30.1	29.0	30.9
Average	do	11.5	12.8	14.4	18.2
Hard Material:					
Number of samples		6	22	22	10
Absorption:					
Minimum	percent	1.4	0.5	0.6	1.0
Maximum	do	11.1	11.3	11.3	11.0
Average	do	4.7	2.7	3.0	3.3

Specific Gravity and Absorption

Tests for bulk specific gravity were made on saturated and surface-dry samples using the mason jar pycnometer.[1] This determination does not appear to have any significance in differentiating between hard and soft pieces. Seven samples of hard materials had bulk specific gravities varying from 1.62 to 2.55; five samples of soft materials varied from 1.48 to 2.57; eight samples classed as borderline materials varied from 2.11 to 2.59. Since there is so much overlapping, use of the test for specific gravity to identify hard and soft pieces does not appear feasible.

The absorption test was made by immersing oven-dried samples in water for a period of 24 hours. The samples were then surface-dried with a towel and weighed, and the absorption expressed as a percentage of the dry weight. A summary of the results obtained is given in table 3. In this table, as in others to follow, each size of each material tested is treated as an individual sample. This permits comparisons between the two types of material, hard and soft, for each size of piece. Although the average values for the two types of material differ markedly, there is some overlapping in the test values. In the 1- to ¾-inch size, for example, one sample of soft material had an absorption of only 3.2 percent, whereas a sample of hard material had a quite high absorption of 11.3 percent. Because of this overlapping, it is not possible to set a value separating hard from soft material and this method is of little use for the identification of soft particles.

Impact Tests

A number of different types of impact tests were used in this investigation. The one which is believed to have the most promise is the test for the toughness of gravel.[2] As shown in figure 1, the apparatus used in this test consists essentially of a 2½-inch steel ball mounted on a steel block to serve as an anvil, and another steel ball of the same size which can be lifted to a maximum height of 7 inches and allowed to fall on the specimen under test. During the course of this investigation, certain improvements in the original apparatus were made so that the ball could be dropped the exact distance desired with but one measurement of the thickness of the specimen under test.

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[1] Described in *Principles of Highway Construction as Applied to Airports, Flight Strips, and other Landing Areas for Aircraft*, Bureau of Public Roads, June 1943. p. 297.

[2] Method T-6-27, *Standard Specifications for Highway Materials and Methods of Sampling and Testing, 1938;* American Association of State Highway Officials, p. 152. This method has been withdrawn by the Association.

In making this test, the specimen was held on the anvil in its most stable position, usually with the least dimension vertical, and the movable ball dropped on the specimen from a height of 1 inch. The height of fall was increased 1 inch after each blow until the specimen failed or until it withstood a drop of 7 inches. Under normal conditions, a test sample containing 50 to 100 pieces of the same sieve size was used. An empirical value for each sample was obtained by multiplying the number of pieces which failed by the square of the drop, in inches, at which they failed, and dividing the sum of these values by the total number of pieces tested. For the purpose of this computation, pieces which did not fail at a drop of 7 inches were assumed to fail at a drop of 10 inches. This value, called the toughness factor, can vary from a minimum of 1 to a maximum of 100.

The results of these tests are shown in table 4. Although higher toughness factors were found for the hard materials than the soft, there is a considerable amount of overlapping of the two sets of values. It is apparent that the toughness test for gravel does not separate hard from soft material. After a review of the detailed records of this test, it appears that the falling ball is much too heavy to separate soft pieces from hard but brittle pieces. Furthermore, when the brittle piece fails, the fracture is apparent, but many pieces of soft material may fail without the break being seen from above as the operator would normally view the specimen. This results in the soft piece being given a higher rating than it should have, thereby decreasing the value of the test.

In an attempt to correct these difficulties, another toughness apparatus was made using a steel ball of 1⅞-inch diameter. For certain sizes of particles, the 1⅞-inch ball was found to furnish more indicative test results than the 2½-inch ball, but the conclusion was reached that to test aggregate of a complete range in size from 2- or 2½-inch to ⅜-inch, at least three toughness testers of different sizes should be used, and each size used to test material of a definite and narrow range in sieve sizes. As time to develop these testers and correlate the test results of each was not available, further consideration of the use of this type of apparatus was deferred.

Table 4.—*Use of the toughness test (2½-inch ball) for the identification of hard and soft materials*

Item	Size of pieces			
	2- to 1½-inch	1½- to 1-inch	1- to ¾-inch	¾- to ½-inch
SOFT MATERIAL				
Number of samples	3	28	29	20
Toughness factor:				
Minimum	18.0	2.7	2.0	1.0
Maximum	59.0	46.2	16.9	5.7
Average	41.6	20.5	7.9	2.5
HARD MATERIAL				
Number of samples	7	23	23	10
Toughness factor:				
Minimum	5.0	4.5	2.9	1.1
Maximum	79.0	67.2	32.3	10.3
Average	42.3	29.8	11.4	3.8

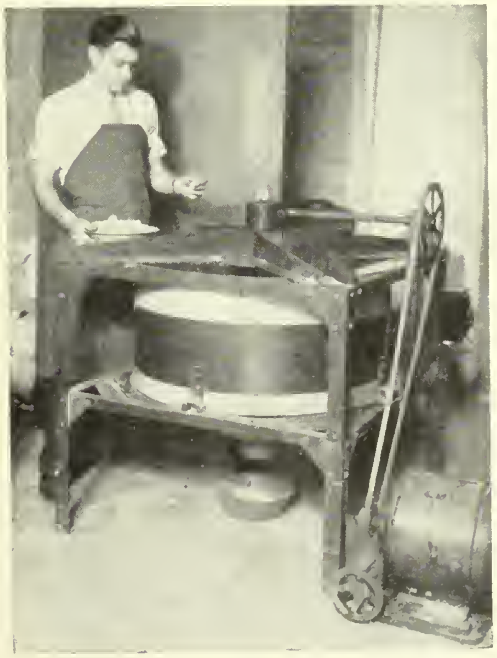
Figure 2.—The rotary soft-stone machine.

Rotary Soft Stone Machine

Another type of impact test tried involves the use of the rotary soft stone machine shown in figure 2. This machine consists essentially of a cast-iron disk revolving in a horizontal plane inside a vertical steel drum made of ⅜-inch steel plate. The disk is 29 inches in diameter and the drum has an inside diameter of 33¼ inches. Ribs on the upper surface of the disk form pockets to catch the material fed on the disk through a sheet-metal cone, and to throw these pieces against the steel drum. Another cone below the disk serves to collect the sample and lead it to a pan in which the tested material can be inspected. The disk is powered by a variable-speed motor and can be operated at speeds from 110 to 200 r.p.m. In these tests, the fastest speed was used. The test was conducted on individual sizes of aggregate. The weight of the test sample was determined, and the pieces passed singly through the machine. After all pieces had been tested, the sample was sieved on the original retaining sieve, and the weight of material passing this sieve expressed as a percentage of the original weight of the sample.

The results obtained with the rotary machine are given in table 5. These results indicate the same conditions as were found for the falling-ball apparatus. Hard but brittle material as well as soft material is readily broken in the rotary machine, and it does not appear possible to separate truly soft pieces from others with this type of test.

Hand-Hammer Tests

Several other types of impact tests were tried. In former years, a hammer test was occasionally used to identify soft pieces. Although the application of a hand hammer involves a number of intangible conditions, it was believed desirable to include this test in the investigation. The hammers used varied from a 2-ounce tile-setter's hammer to a stone-mason's hammer with a 2-pound head, as shown in figure 3. Several different types of technique were used. In one, the hammer was allowed to drop without bending of the wrist; in another, the hammer was swung by movement of the wrist only; in a third, the end of the hammer handle was placed on the table and the hammer head allowed to fall through an arc of about 8 inches. The tile-setter's hammer has one flat and one sharply beveled end. With this hammer, one method included a free swing of the arm, with the flat face striking the piece under test. In another application of the same small hammer, the beveled end was used to peck at the test specimen to determine whether or not the material could be flecked away or cut by

Table 5.—*Use of the rotary soft-stone machine for the identification of hard and soft materials*

Item		Size of pieces			
		2- to 1½-inch	1½- to 1-inch	1- to ¾-inch	¾- to ½-inch
SOFT MATERIAL					
Number of samples		3	25	28	18
Loss:					
Minimum	percent	13.5	2.5	2.5	1.4
Maximum	do	22.0	29.9	21.4	20.8
Average	do	19.0	11.2	10.4	9.6
HARD MATERIAL					
Number of samples		5	19	21	9
Loss:					
Minimum	percent	1.0	2.0	2.5	1.4
Maximum	do	24.0	19.5	32.0	34.2
Average	do	15.9	10.3	9.9	11.6

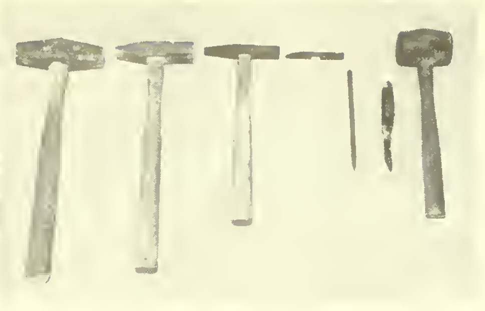

Figure 3.—Hammers and scribes used in soft piece test. Left to right: 2-pound stone-mason's hammer, 1-pound tinner's hammer, 8-ounce tinner's hammer, 2-ounce tile-setter's hammer, brass scratch pencil, knife, and rubber mallet.

light taps. Consideration was also given to the type of failure of a specimen under blows of a reasonably heavy hammer. It was thought that if the material were soft it would crush under the hammer, producing a large amount of powder with relatively few large fragments; hard material, on the other hand, would break to sharp-edged fragments with relatively little powdering. Of all these hand-hammer tests, none gave satisfactory results, and this type of test was discontinued.

Consideration was then given to an impact test using as the striker an article which would deform around the piece under test but still transmit stress to it. The articles considered for this application were a device known in some circles as a blackjack, and a mallet with a rubber head such as is used in removing dents from automobile bodies. A blackjack having a leather case filled with lead shot was tried, but after a few tests the leather broke and further consideration of this was discontinued. For a few very soft materials, the rubber mallet gave satisfactory results, but moderately soft materials of 1-inch size or larger could not be crushed with the mallet. This test was also discontinued.

Compression Tests

The suitability of a compression test to identify soft pieces was tried. A small apparatus known as the Douglass stone meter[3] has been used for such a test, but the maximum load specified is only 75 pounds and this load will crush only the weakest specimens. This apparatus is shown in figure 4. The use of a hydraulic compression testing machine was then proposed. In the method considered, an attempt was made to apply load to the particle through ½-inch steel balls. For pieces of irregular shape, this was very difficult to perform, and flat-faced steel cylinders of ½-inch diameter were substituted for the balls. The pieces under test were ground on a lap to secure bearing faces, and were tested with the least dimension in a vertical position. Load was applied at a slow but predetermined rate and failure determined to the nearest 10 pounds. Each sample tested contained from 20 to 100 pieces.

The results obtained are given in table 6. It will be observed that no well-defined separation between hard and soft materials is obtained. Study of the pieces under load revealed one interesting feature. In testing one material—a shale which occurs in rather thin pieces—it was noticed that the portion of the piece not in contact with the loading cylinders fell away at a relatively low load, but the material between the loading cylinders became more compact as the loading continued. In one case, this material was loaded to 35,000 pounds or almost 180,000 pounds per square inch. It is apparent that this type of loading is unsuitable for thin specimens of soft material and that the test does not separate hard from soft material.

Freezing and Thawing Test

The freezing and thawing test was used to determine whether any relationship could be established between the results of this test and the hardness of the materials considered. The samples were immersed in water for 24 hours prior to freezing, frozen at about 15° F. in water, and thawed at about 80° F. A 24-hour cycle was used. After 10 cycles of the test, the samples were examined. With only a few exceptions, this short test failed to differentiate between hard and soft materials. The test was then resumed for another 10 cycles. After the twentieth cycle, the samples were dried, sieved on a sieve with openings having linear dimensions one-half the size of those in the original retaining sieve, and the material passing the sieve expressed as a percentage of the weight of the original sample. The half-size sieves were used rather than the original sieves to prevent the inclusion, in the loss, of those pieces which might pass the original retaining sieve due only to minor flaking or chipping. Results of these tests are given in table 7.

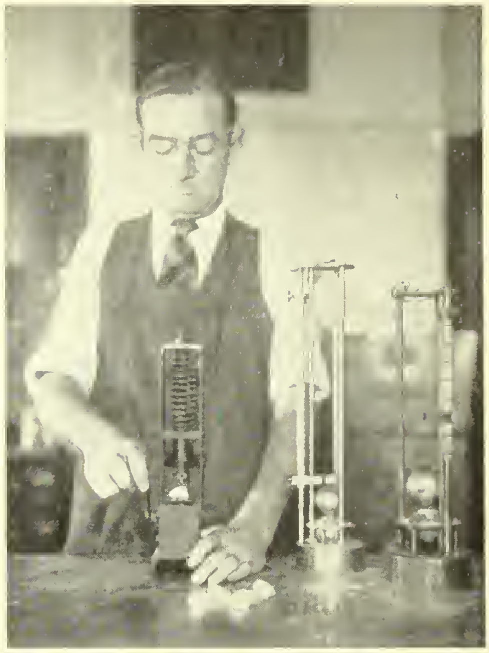

Figure 4.—Douglass stone meter in use. To the right are the toughness machines with the 1⅞-inch and 2½-inch balls.

Table 6.—*Use of the compression machine for identification of hard and soft materials (load applied through flat faces of ½-inch diameter)*

Item		Size of pieces			
		2- to 1½-inch	1½- to 1-inch	1- to ¾-inch	¾- to ½-inch
SOFT MATERIAL					
Number of samples		4	9	20	8
Compressive strength:					
Minimum	lb	1,130	540	120	120
Maximum	do	2,910	2,970	2,770	3,530
Average	do	1,840	1,270	860	1,010
HARD MATERIAL					
Number of samples		5	10	13	3
Compressive strength:					
Minimum	lb	500	860	690	480
Maximum	do	3,460	2,950	3,250	1,290
Average	do	1,890	1,830	1,750	860

Table 7.—*Use of freezing and thawing (20 cycles) for identification of hard and soft material*

Item		Size of pieces			
		2- to 1½-inch	1½- to 1-inch	1- to ¾-inch	¾- to ½-inch
SOFT MATERIAL					
Number of samples		1	12	21	11
Loss passing ½-size sieve:					
Minimum	percent		3.1	5.0	9.4
Maximum	do		92.8	93.0	92.2
Average	do	9.2	26.1	42.5	51.5
HARD MATERIAL					
Number of samples		4	11	14	3
Loss passing ½-size sieve:					
Minimum	percent	2.7	0.6	0.4	5.4
Maximum	do	31.6	18.2	51.4	25.3
Average	do	11.8	5.0	11.4	13.1

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[3] Douglass stone meter described in Method T-8-24. *Standard Specifications for Highway Materials and Methods of Sampling and Testing, 1938;* American Association of State Highway Officials, p. 154. This method has been withdrawn by the Association.

Considerable overlapping of test results is found, but the average values are separated so widely that they indicate some possibilities in the use of this method for the identification of hard and soft aggregates. The main objections to this test are the length of time required and the expense and nonportability of the apparatus. The test appears to be of some value for use in research investigations, but it hardly seems suitable for a routine testing procedure.

Revision of the Compression Test

Shortly after this investigation had been started, the entry of the United States into World War II required that the work be discontinued. Due to lack of storage space, only a few of the samples originally received were kept intact and, consequently, a suitable correlation between the two portions of the investigation cannot be established without repeating a large amount of work.

When the investigation was resumed in 1945, a study of the then existing data indicated that possibly a fair trial had not been given to the compression test. In the previous work, plane and approximately parallel faces had been ground on each piece tested, and the pieces loaded between flat-faced cylinders of ½-inch diameter. The labor of grinding faces on 50 to 100 pieces of each size in each sample tested was considered objectionable, and some means of testing pieces of any shape without preliminary grinding was sought. Trials were made of a number of different types of loading devices. After considerable work, a loading device consisting of a single-point upper contact and a three-point lower support was adopted. As shown in figure 5, the upper contact consists of a short steel rod of ½-inch diameter, with a hemispherical end, which is fastened to the spherical bearing block of a hydraulic cube-testing machine. The lower bearing consists of three ½-inch diameter steel balls grouped together so that their surfaces are in contact, and welded to a small steel base. With this loading and support arrangement, most pieces of both regular and irregular shape can be placed in a stable position for the test.

To obtain values for use in identifying pieces of hard, sound aggregate, several hundred pieces of each size of three different materials were tested and the breaking load of each piece determined. From the data obtained, the following loads were selected as indicating hard materials:

Size of piece, inches	*Load, pounds*
⅜ to ½	200
½ to ¾	350
¾ to 1	500
1 to 1½	750
1½ to 2	1,100
2 to 2½	1,500

Tests of Commercial Gravels

With the establishment of these tentative acceptance values for hard material, it was believed desirable to conduct tests on a number of different materials to determine whether the test would be found satisfactory. Only a few of the materials obtained in 1941 were still available, and these had been sampled so extensively that complete ranges in size of particle could not be obtained. To obtain samples for further tests, the National Sand and Gravel Association was requested to furnish samples of commercially produced gravels which would contain some soft material. To the samples received from the Association were added a number of gravels which had been submitted to the laboratory for routine tests. These samples were tested for soft-piece content using the compression test described above, the 20-cycle freezing and thawing test, and the Los Angeles abrasion test. In the last test, determinations of the percentage of wear were made at 100 and 500 revolutions and a ratio of the losses used as an expression of the amount of soft material in the sample.

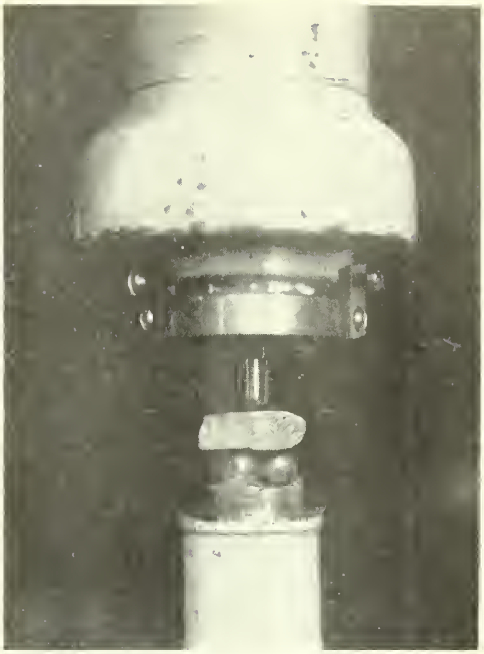

Figure 5.—Compression test for soft pieces, using a three-point support and a single-point load-bearing contact.

Table 8.—*Soft-piece tests on commercially produced gravels*

Sample No.	Los Angeles abrasion test: Loss at 100 revolutions	Los Angeles abrasion test: Loss at 500 revolutions	Los Angeles abrasion test: Ratio of losses [1]	Compression test: Percentage of soft material, by weight	Freezing and thawing test: Loss after 20 cycles
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
67464	6.0	28.1	21.4		27.4
67621	4.9	24.2	20.2		10.4
67622	4.4	22.7	19.4		10.3
67628	8.1	32.6	24.8		9.1
67629	5.4	27.3	19.8		9.3
67680	4.2	23.4	17.9		6.8
67696	4.6	23.8	19.3		7.5
67757	5.6	23.6	23.7	12.1	4.2
67858	7.3	26.4	27.7		11.0
67909	5.6	23.3	24.0		17.0
67910	4.2	25.0	16.8		17.2
67954	6.1	27.8	21.9		20.4
68516	5.6	26.5	21.1	15.6	11.2
68768	7.6	30.7	24.8	24.5	19.7
68769	5.4	26.3	20.5	19.5	6.5
68770	5.7	27.1	21.0	21.3	5.7
68771	4.1	21.3	19.2	14.4	7.6
68772	6.5	29.0	22.4	26.2	8.4
68773	5.3	26.6	19.9	23.7	3.8
68774	8.5	32.9	25.8	33.1	12.7
68775	6.8	30.8	22.1	18.8	8.9
68776	6.4	31.8	20.1	38.9	21.3
68777	7.4	32.4	22.8	29.8	29.2
68778	6.8	29.6	23.0	14.0	4.1
68779	5.8	25.5	22.8	23.7	2.4
68780	6.6	30.3	21.8	17.6	9.5

[1] Ratio of loss at 100 revolutions to loss at 500 revolutions.

The results of these tests are given in table 8. Inspection of the results discloses immediately that there is little agreement among the indications of the three methods of test. For some few samples, all three methods show the presence of appreciable amounts of soft material. In most cases, however, the three methods give different results. One reason for nonuniformity among the three methods is the variance of the unit of failure on which the test result depends. In the compression test, the pieces of the sample are tested separately, and the whole piece is discarded if it fails to meet the conditions of the test. In the Los Angeles abrasion test and in the freezing and thawing test, it is entirely possible that only a portion of a given particle may be included in the loss. Consequently, it is doubtful that any satisfactory agreement among different methods of test can be established unless the unit of failure is the same in each case.

Consideration of the values obtained in the compression test shown in table 8 indicates that the limits set for the identification of hard pieces are probably too severe. A review of the individual test results for each sample shows a very high percentage of failures in the smaller sizes of piece, and also an excessive percentage of failures for pieces which tend toward a flat shape. In this test, the point of application of the load is normally above a point equidistant from the point of support of each of the three balls in the lower bearing. With thin or flat pieces, this could result in flexure of the piece under test, and the failure would be by bending instead of by compression.

Review of Methods of Test

Late in 1946, several new methods of testing aggregates for content of soft pieces were suggested. These new methods included a scratch test using a scribe of yellow brass, and two abrasion tests. In one of the latter, the sample was placed in a canvas bag with a charge of steel balls and the bag dropped on or swung against an anvil a given number of times. In the other abrasion test, the sample alone was placed in a bag and subjected to blows delivered by a rubber-headed mallet, freely swung as shown in figure 6. In trying these methods, it was considered desirable to include in the program of tests some of the methods which had previously been used.

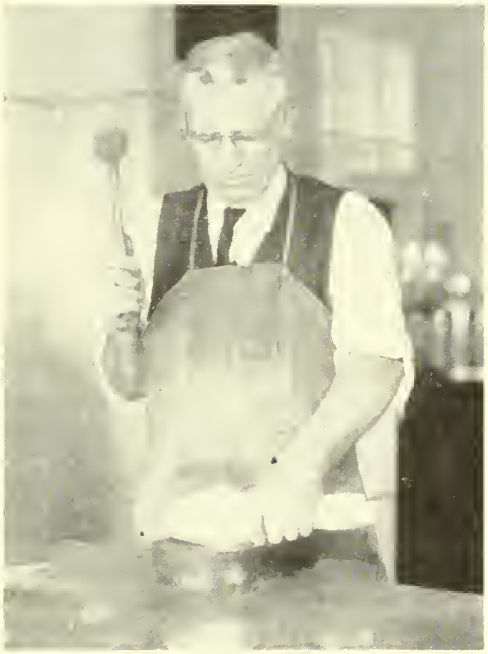

Figure 6.—Bag and mallet test for soft pieces.

The material used in these tests consisted of quartz gravel of a reasonably uniform hardness to which was added definite quantities of soft stone or gravel obtained from the remnants of the samples received in 1941. It seemed possible that the test operators might be influenced by the color of the different pieces and would unconsciously classify them on this basis, and suggestions were made that all the material be dyed to a uniform color, or the operators fitted with colored glasses. Both suggestions, after some consideration, were rejected and the difficulty overcome in part by the use of some soft material of nearly the same color as the base gravel. Furthermore, most of the tests made have definite end points, and the operator is not required to decide whether the material does or does not meet the conditions of the test.

Several changes in the test procedure as previously used were thought to be desirable. A brief description of each method follows.

Method 1: Steel scratch test.—Each piece was scratched with a sharp knife blade. The weight of the pieces identified as soft was reported as a percentage of the original weight of the sample.

Method 2: Brass scratch test.—Each piece was scratched with a pointed scribe prepared from ¼-inch diameter yellow brass rod. The results were reported as in method 1.

Method 3: Light hammer test.—The test piece was placed on an anvil in its most natural position of repose, held firmly with the fingers, and struck at the center of the upper face with the flat end of the tile-setter's hammer. The hammer was swung through an arc of about 6 inches using only a natural movement of the wrist. Softness was recognized by crushing or crumbling of the piece under test. A clean fracturing or splintering of the piece—that is, breaking into smaller but solid fragments without considerable powdering—was not considered indicative of softness. More than one blow was permitted if necessary to classify the piece properly. The results were reported as in method 1.

Method 4: Rubber mallet test.—This method followed the procedure given in method 3, but using a rubber mallet instead of the tile-setter's hammer.

Method 5: Toughness test for gravel.—The falling-ball apparatus previously described, and illustrated in figure 1, was used. The 2½-inch ball was dropped as follows:

Size of piece, inches	*Height of fall, inches*
1½ to 1	3
1 to ¾	2
¾ to ½	1

The test piece was held on the anvil and adjusted under light tapping of the free ball to a secure bearing. The free ball was then raised to the height indicated for the size of aggregate under test and allowed to fall on the piece. Softness of the piece was shown by crushing, powdering, or crumbling. The test result for each piece was reported as in method 1.

Method 6: Toughness test for gravel.—The procedure given in method 5 was repeated, using the apparatus containing the 1⅞-inch ball, and with heights of fall as follows:

Size of piece, inches	*Height of fall, inches*
1½ to 1	5
1 to ¾	3
¾ to ½	1

Method 7: Rotary soft-piece test.—The rotary machine previously described, and illustrated in figure 2, was operated at its slowest speed (about 110 r.p.m.). The pieces of the test sample were fed separately into the machine and all debris caught. The material passing the original retaining sieve was determined, and expressed as a percentage, by weight, of the original sample.

Method 8: Douglass stone meter test.—The stone meter was used with the following loads:

Size of piece, inches	*Applied load, pounds*
1½ to 1	125 (Maximum loading of spring)
1 to ¾	75
¾ to ½	60

The results were reported as in method 1.

Method 9: Compression test.—The tests were made in the hydraulic testing machine with the one-point upper and three-point lower bearing surfaces, using the following loads:

Size of piece, inches	*Applied load, pounds*
1½ to 1	500
1 to ¾	350
¾ to ½	200

The piece tested was placed in its most stable position on the three-point support, and load applied without shock through the single-point contact on the upper surface of the specimen. Breakage of the specimen at a load below those indicated constituted failure. The results were reported as in method 1.

Method 10: Bag abrasion method.—A 1,000-gram sample was placed in a 10- by 14-inch canvas bag with an abrasive charge of five 1⅞-inch diameter cast-iron or steel balls. The neck of the bag was fastened securely to prevent loss of the sample. The bag was swung 100 times through a distance of 12 inches against an anvil. The sample was then removed from the bag and sieved on a No. 4 sieve. The material passing this sieve was expressed as a percentage of the original weight of the sample.

Method 11: Bag and mallet method.—A 1,000-gram sample was placed in a 6- by 9-inch canvas bag and the neck fastened securely to confine the sample in the least possible space. The bag was then placed on an anvil and struck 100 times with a rubber-headed mallet. The blows were distributed over the side of the bag, and after each tenth blow the bag was turned over to expose the lower side. The results were determined as in method 10.

Method 12: Los Angeles abrasion test.—Samples weighing 5,000 grams of each size of material were tested in the Los Angeles machine, with an abrasive charge of 12 steel balls weighing 5,000 grams. The percentage of wear was determined at 100 and 500 revolutions.

The results obtained in these tests are given in table 9. The values shown in the first line of the table are the percentages, by weight, of soft material which were added to the quartz gravel. The values given in the rest of the table, except for those of the Los Angeles abrasion tests, are the percentages by weight of soft material found by the test method indicated. The values given for the Los Angeles tests are the losses in percentages by weight obtained in the tests. The ratio of the losses at 100

Table 9.—***Results of tests for soft pieces using prepared samples***

Method	Percentage of soft pieces			
	Size of pieces			Average
	1½- to 1-inch	1- to ¾-inch	¾- to ½-inch	
Soft pieces placed in material	4.8	16.2	10.4	10.5
Scratch hardness methods:				
Steel scribe	17.6	18.2	14.9	16.9
Brass scribe	4.8	16.3	10.6	10.5
Impact methods:				
2-ounce hammer	11.3	18.8	17.9	16.0
Rubber mallet	1.4	10.1	12.6	8.0
Rotary machine	14.6	10.6	10.9	12.0
Toughness, 2½-inch ball	10.2	34.1	21.1	21.8
Toughness, 1⅞-inch ball	20.1	22.0	10.2	17.4
Compression method:				
Douglass machine	8.5	4.8	0.6	4.6
3-point support	4.2	16.9	21.0	14.0
Abrasion method:				
Bag abrasion	6.7	18.0	14.0	12.9
Bag and mallet	6.4	11.7	11.7	9.9
Los Angeles machine:				
Loss at 100 revolutions	5.1	11.9	14.8	—
Loss at 500 revolutions	22.5	40.3	45.3	—
Ratio, 100/500	22.7	29.5	32.7	—

and 500 revolutions has been thought to have some relation with the amount of soft material in the sample. A comparison between these values and the amounts of soft material actually added to the base gravel indicates that this test, as made on individual sizes of aggregate, does not furnish test results which are indicative of the amount of soft material in the sample.

The most favorable results obtained in this series of tests are those furnished by the brass scribe, the rotary soft-piece machine, and the bag and mallet test. In previous work, when the rotary machine was operated at a speed of 200 r.p.m., it had not given very significant results. Although much better results are obtained when the machine is operated at a slower speed, further consideration of this type of testing equipment was discontinued as the machine is not suitable for field use.

Bag and Mallet Test

Some questions developed regarding the size of sample which would be most desirable in the bag and mallet method, and also how many tests should be made to obtain a test value representative of a given material. From a practical consideration, a sample of the size used appears to be about as large as is desired for readiness of handling. Should the material to be tested have a maximum size greater than 1½ inches, the size of the test sample must be increased and a larger bag be used. Considerable development work along this and other lines remains to be done before this method can be adopted for use. To determine the other feature of particular interest here—that is, the number of 1,000-gram samples which should be tested for a material with a maximum size of 1½ inches—another series of tests was made.

In these tests, a soft sandstone was added in definite quantities to a quartz gravel to prepare four aggregates containing 0, 4, 8, and 12 percent soft material. Each material, both the hard and soft, was graded uniformly from 1½- to ⅜-inch. After each aggregate had been prepared, it was mixed thoroughly and ten 1,000-gram samples taken for test. The attempt made here was to duplicate conditions which would exist if a sample of graded aggregate were tested by the bag and mallet method. Each sample was placed in a double canvas bag, one bag inside of the other, and struck 100 times with a rubber-headed mallet. The sample was then sieved on a No. 4 sieve, and the amount passing the sieve determined as well as the amount of soft material retained on the sieve.

Table 10.—***Bag and mallet tests on prepared samples of quartz gravel containing soft sandstone***

Sample No.	Amount of soft stone added to gravel							
	None		4 percent		8 percent		12 percent	
	Loss	Cumulative average loss	Loss	Cumulative average loss	Loss	Cumulative average loss	Loss	Cumulative average loss
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	4.4	4.4	6.2	6.2	12.5	12.5	9.7	9.7
2	3.7	4.0	6.2	6.2	9.9	11.2	11.2	10.4
3	5.9	4.7	6.3	6.2	8.7	10.4	12.3	11.1
4	4.3	4.6	7.4	6.5	10.6	10.4	10.7	11.0
5	3.7	4.4	5.8	6.4	8.2	10.0	13.1	11.4
6	3.9	4.3	7.0	6.5	9.0	9.8	12.4	11.6
7	4.8	4.4	6.0	6.4	10.0	9.8	13.4	11.9
8	4.7	4.4	6.3	6.4	6.9	9.5	9.8	11.6
9	5.0	4.5	6.5	6.4	6.9	9.2	10.9	11.5
10	3.7	4.4	5.0	6.3	9.5	9.2	9.1	11.3

A summary of the results obtained is given in tables 10 and 11. In table 10, the percentage of loss for each sample tested is given, together with a cumulative average. For most aggregates, an average value obtained from tests of three samples is very nearly the same as the average for all ten samples of a kind tested. Of more importance, however, is the comparison of the amount of soft stone placed in the aggregate, the test result obtained, and the amount of soft stone remaining in the sample after the test, as shown in table 11. Tests of the base gravel show a loss of 4.4 percent. The results obtained in tests of the gravel containing 4 percent of soft stone show 6.3 percent loss but 2.0 percent of soft stone is left in the sample. This 6.3 percent loss then includes 2.0 percent of soft stone and 4.3 percent from the quartz gravel.

Table 11.—***Percentage of soft stone remaining in sample after test.[1] and loss from gravel***

Soft stone added to gravel	Cumulative average loss	Soft stone remaining	Loss from gravel
Percent	*Percent*	*Percent*	*Percent*
0	4.4		4.4
4	6.3	2.0	4.3
8	9.2	3.3	4.5
12	11.3	4.5	3.8

[1] Averages for 10 samples.

By a similar method of figuring, the test results for the aggregates containing 8 and 12 percent of soft stone include 4.5 and 3.8 percent, respectively, of the quartz gravel in the loss. Although it is granted that the quartz gravel may contain material of a friable nature, a comparison of the gravel and the sandstone used in these tests indicates that a satisfactory method of test should show a greater recovery of the soft material in the test results. The fact that from 38 to 50 percent of the soft sandstone placed in the sample remains there after the test shows definitely that the test as made is not satisfactory.

Brass Scratch Test

In this investigation, considerable difficulty was encountered in trying to unify the conceptions offered by different authorities regarding a proper description of a soft piece, and much time and effort were spent to develop a satisfactory method of test for soft pieces. In addition to the various tests described here, many more fanciful tests were considered and, in some cases, plans for quite elaborate pieces of testing equipment were prepared. A few of these fanciful tests were tried but did not prove even remotely satisfactory. Possibly we have been trying to make something difficult of a really simple problem: The testing of aggregates to determine the presence and quantity of soft pieces—those which yield easily to physical pressure or are not resistant to cutting or wear.

Among all of the test methods considered, the most simple and direct is a scratch hardness test. This would not indicate the pieces which are unsound, or light-weight, or highly absorptive; or the pieces of chert which appear to be included by some authorities in the general classification of soft pieces. It would, however, show which pieces of the sample are actually soft, including those formed of a soft material and those which are so poorly bonded that the separate particles in the piece are easily detached from the mass.

A satisfactory test for scratch hardness was used in this investigation. It consists merely of scratching the material under test with a piece of yellow brass, as shown in figure 7. This brass will not scratch limestone of good quality, but it is hard enough to scratch badly weathered materials which may be objectionable for use in concrete. The brass used first in these tests consisted of a $\frac{1}{4}$-inch rod which has a Rockwell hardness of about B70. Later in this work, the thought of preparing a pencil with a brass rod replacing the lead was developed. For this purpose, drill-rod brass[4] of about 1/16-inch diameter was used. Efforts were made to obtain a Rockwell hardness value for this material, but the rod was too small to secure a satisfactory reading. For the purpose of this test, it is probable that minute distinctions in the hardness of the brass used is of little moment. Possibly it is sufficient to describe the material as hard, yellow brass.

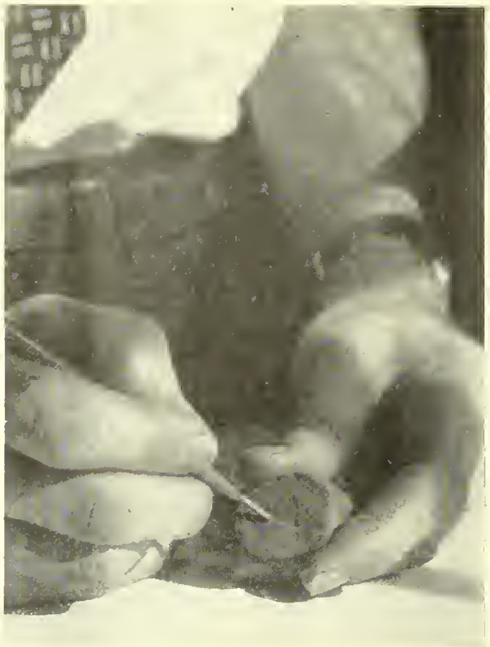

Figure 7.—The scratch test, using a hard, yellow brass scribe "pencil" made with a drill-rod brass core encased in wood. This is the only test for soft aggregates suitable to both field and laboratory use.

The test consists of separating the aggregate into different sizes, down to $\frac{3}{8}$-inch, and determining the scratch hardness of a representative number of pieces of each size. With material of fairly uniform quality, 10 pieces of a size may be sufficient; but 50 or 100 pieces of a size may be required for heterogeneous materials. The weight of the pieces identified as soft is determined for each size, and a weighted average based on the grading of the sample is computed.

This test is for soft pieces only. If it is desired to limit the amount of other types of deleterious materials in aggregate, separate mention of these should be made in specifications.

The courtesy of the State highway departments and the National Sand and Gravel Association in furnishing samples for use in these tests is appreciated, as are the many valuable suggestions offered by Mr. T. R. Smith, and the assistance furnished by him and the other employees of the laboratories of the Bureau of Public Roads.

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[4] So called as it is obtainable in the same sizes as steel twist drills.

Errata

In order to rectify a printer's error, page 121 of the February 1951 issue of PUBLIC ROADS (Vol. 26, No. 6) was reprinted with a heading "Errata Sheet" and distributed to all subscribers. A number of inquiries since received indicate that the purpose of this was in some cases misunderstood. The error involved a mix-up of text on page 121, under the caption "Part IV—Over-All Intersection Capacity." As originally printed, the first and third columns were inadvertently interchanged, so that the lower portion of the page must be read (by columns) from right to left.

The printer also unfortunately turned chart 4, on page 128, to read from the left edge of the page instead of the right edge, thus considerably impairing the facility of use of the chart series.

While these errors occurred after final proofing and were beyond the control of the editorial staff of PUBLIC ROADS, they nevertheless greatly regret the inconvenience caused thereby.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

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Highway Needs of the National Defense (House Document No. 249). 50 cents.
Highway Practice in the United States of America. 50 cents.
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ANNUAL REPORTS

(See also adjacent column)

Public Roads Administration Annual Reports:
1943. 1944. 1945.

MISCELLANEOUS PUBLICATIONS

Bibliography on Automobile Parking in the United States.
Bibliography on Highway Lighting.
Bibliography on Highway Safety.
Bibliography on Land Acquisition for Public Roads.
Bibliography on Roadside Control.
Express Highways in the United States: a Bibliography.
Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF FEBRUARY 28, 1951

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM: PROGRAMMED ONLY			PLANS APPROVED CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$18,088	$10,575	$5,344	214.9	$6,542	$3,327	129.5	$14,783	$7,107	352.5	$31,900	$15,778	696.9
Arizona	274	7,709	5,445	186.0	353	243	2.0	6,610	4,734	120.1	14,672	10,422	308.1
Arkansas	6,990	8,515	4,709	238.2	3,073	1,703	97.0	15,321	7,582	404.8	26,909	13,994	740.0
California	6,040	57,266	17,167	273.3	9,774	4,948	44.2	52,113	24,768	262.2	119,153	46,883	579.7
Colorado	3,957	6,231	3,527	223.6	2,921	1,522	64.0	11,152	6,193	241.3	20,304	11,242	528.9
Connecticut	3,668	7,664	3,888	21.3	3,659	2,025	1.7	6,353	3,485	10.6	17,676	9,398	33.6
Delaware	2,185	1,014	608	22.5	19	8		6,449	3,153	38.4	7,482	3,769	60.9
Florida	7,862	8,222	4,339	153.4	9,242	4,496	176.6	17,707	8,755	392.1	35,171	17,590	722.1
Georgia	8,160	12,985	6,668	243.0	11,898	5,071	91.9	28,555	14,135	694.4	53,438	25,874	1,029.3
Idaho	5,583	8,839	5,579	271.6	2,530	1,754	121.1	6,116	3,260	122.1	17,485	10,593	514.8
Illinois	21,364	53,518	28,307	506.5	24,055	12,443	220.8	47,441	22,880	277.5	125,014	63,630	1,004.8
Indiana	13,154	37,044	18,421	183.7	4,648	2,329	28.6	18,424	9,353	96.2	60,116	30,103	308.5
Iowa	3,388	14,498	6,810	750.9	10,284	5,090	301.7	11,310	5,731	291.7	36,092	17,631	1,344.3
Kansas	7,721	11,011	5,262	1,044.8	5,601	2,801	376.9	8,376	4,187	493.8	24,988	12,250	1,915.5
Kentucky	3,005	21,989	10,846	264.3	3,364	1,686	57.1	17,456	8,616	302.0	42,809	21,148	623.4
Louisiana	7,656	14,456	6,870	111.4	8,463	3,984	55.5	21,533	11,363	242.4	44,452	22,217	409.3
Maine	4,280	5,216	2,764	65.6	1,697	889	7.2	6,659	3,538	68.7	13,572	7,191	141.5
Maryland	5,296	4,576	2,265	23.2	3,822	1,481	14.1	11,138	5,099	39.7	19,536	8,845	77.0
Massachusetts	3,677	14,879	6,640	7.3	1,458	529		65,113	32,328	61.9	81,450	39,497	69.2
Michigan	11,003	18,424	9,121	524.4	11,780	6,377	207.9	39,759	16,197	275.2	69,963	31,695	1,007.5
Minnesota	6,034	15,689	8,219	1,382.4	3,075	1,963	114.9	16,791	8,845	276.8	35,555	19,027	1,774.1
Mississippi	9,373	9,109	4,747	345.7	8,161	4,290	229.9	5,871	2,870	178.3	23,141	11,907	753.9
Missouri	12,956	23,668	12,429	690.2	14,338	7,752	351.6	27,541	13,985	369.5	65,547	34,166	1,411.3
Montana	10,106	13,472	6,887	443.4	3,721	2,197	80.5	9,501	5,703	202.5	26,694	14,787	726.4
Nebraska	8,635	16,585	8,628	571.4	7,033	3,479	217.8	9,298	4,947	253.8	32,916	17,054	1,043.0
Nevada	5,738	2,271	1,873	33.0	928	775	65.3	3,496	2,883	113.8	6,695	5,531	212.1
New Hampshire	3,145	2,591	1,472	18.9	443	210	3.7	4,555	2,263	39.4	7,589	3,945	62.0
New Jersey	6,793	5,892	2,946	7.4	3,048	1,522	4.2	18,862	8,702	21.6	27,802	13,170	33.2
New Mexico	4,513	4,376	2,800	88.8	857	552	18.7	9,416	6,008	262.8	14,649	9,360	370.3
New York	47,223	56,400	28,998	153.5	17,012	7,798	42.9	106,555	49,795	200.3	179,967	86,591	396.7
North Carolina	8,692	11,096	5,439	278.0	8,626	3,932	147.2	22,995	11,087	529.9	42,717	20,458	955.1
North Dakota	5,020	8,465	4,371	1,254.2	5,216	2,616	405.0	4,849	2,403	408.1	18,530	9,390	2,067.3
Ohio	15,836	20,415	9,561	222.9	18,376	10,175	203.9	71,424	35,484	267.3	110,215	55,220	694.1
Oklahoma	2,077	21,834	10,268	384.4	6,808	3,575	83.7	24,068	11,815	440.2	52,710	25,658	908.3
Oregon	2,302	3,823	2,238	56.2	7,653	3,922	78.2	10,555	6,094	158.4	22,031	12,254	292.8
Pennsylvania	21,010	7,825	3,823	15.8	11,314	5,510	15.7	80,533	39,803	227.7	99,672	49,136	259.2
Rhode Island	1,769	6,340	3,170	45.0	1,090	544	7.1	13,175	6,691	10.0	20,605	10,405	62.1
South Carolina	5,396	8,462	4,370	227.5	2,877	1,440	58.2	7,663	4,032	216.5	19,002	9,842	502.2
South Dakota	1,154	13,829	7,952	1,126.8	3,240	1,862	231.1	8,378	5,177	610.1	25,447	14,991	1,968.0
Tennessee	7,883	12,273	5,824	271.4	5,393	2,772	101.7	18,975	8,743	323.9	36,641	17,339	697.0
Texas	13,858	8,409	4,292	196.3	15,256	7,717	167.4	46,889	22,196	895.8	70,554	34,205	1,259.5
Utah	4,903	3,089	2,259	71.0	1,488	1,102	36.9	4,484	3,250	140.4	9,061	6,611	248.3
Vermont	1,006	4,577	2,410	63.2	891	449	8.1	4,236	2,077	28.7	9,704	4,936	100.0
Virginia	10,160	19,631	9,727	396.6	5,910	2,945	197.1	14,249	6,943	238.6	39,790	19,615	832.3
Washington	6,520	6,893	2,446	71.5	1,602	836	19.0	21,113	10,112	127.3	29,608	13,394	217.8
West Virginia	5,316	13,211	5,242	92.1	4,512	2,257	50.1	9,212	4,659	96.7	26,935	12,158	238.9
Wisconsin	9,295	28,031	14,698	762.6	3,907	1,889	94.5	13,113	6,498	313.4	45,051	23,085	1,170.5
Wyoming	1,530	1,772	1,151	28.7	4,141	2,694	109.4	6,082	3,745	163.0	11,995	7,590	301.1
Hawaii	2,296	7,716	3,336	14.3	2,193	781	10.6	6,138	2,498	18.5	16,047	6,615	43.4
District of Columbia	3,951	3,960	1,980	4.1	166	83	.9	1,557	971	.9	5,683	3,034	5.9
Puerto Rico	3,635	13,161	6,027	66.0	855	377	3.5	8,128	3,778	33.7	22,144	10,182	103.2
TOTAL	391,476	699,496	344,163	14,713.2	295,313	150,722	5,156.6	1,022,070	506,521	11,955.5	2,016,879	1,001,406	31,825.3

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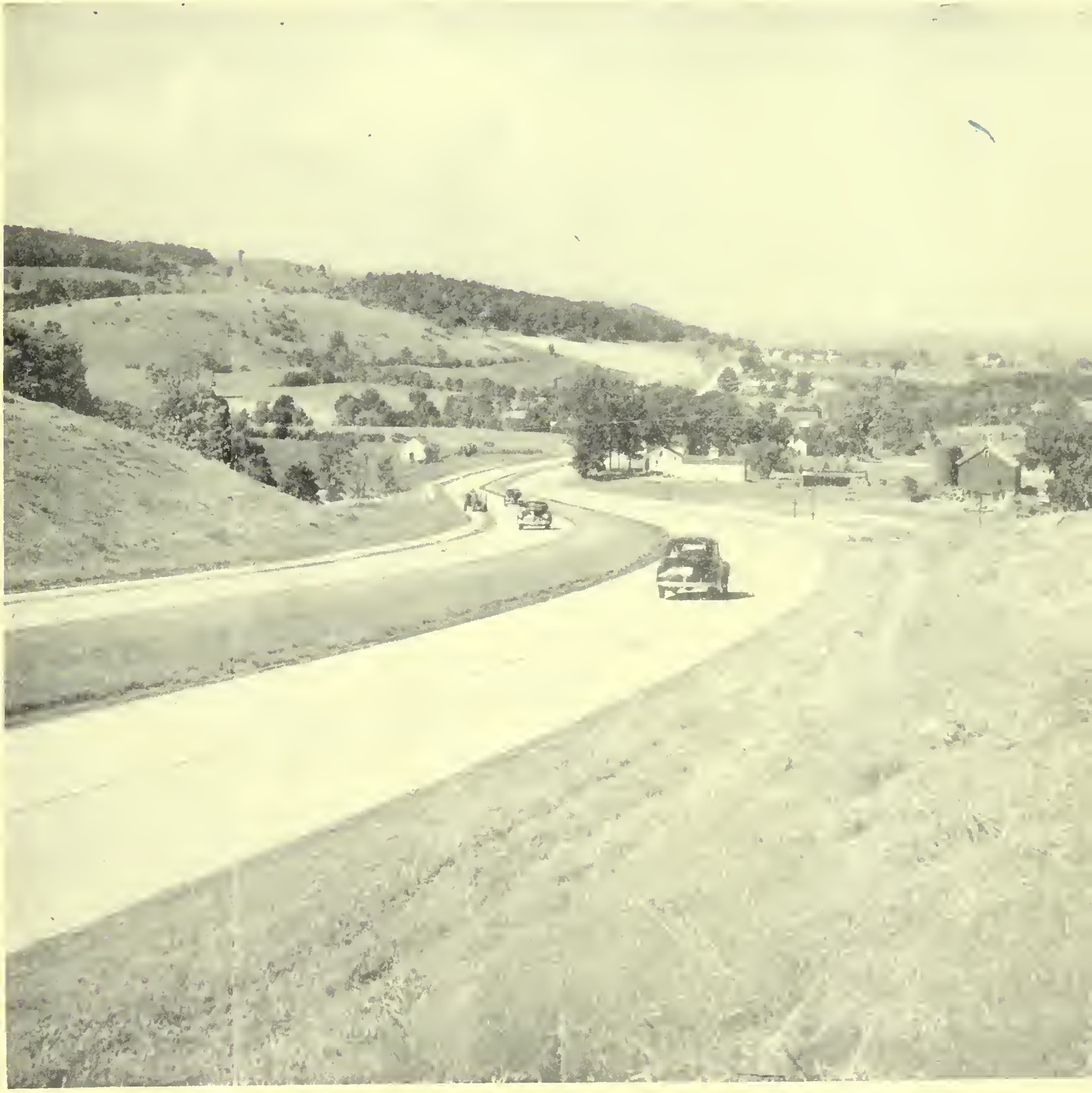
Public Roads

A JOURNAL OF HIGHWAY RESEARCH



An additional cent of gasoline tax, costing the average motorist 13 cents a week, will provide many miles of better highways

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Vol. 26, No. 8 *June 1951*

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The printing of this publication has been approved by the Director of the Bureau of the Budget January 7, 1949.

BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE
E. A. STROMBERG, Editor

Contents of this publication may be reprinted. Mention of source is requested.

The Gasoline Tax in Relation to Automobile Operation and Highway Costs

BY THE RESEARCH REPORTS BRANCH
BUREAU OF PUBLIC ROADS

Reported by E. M. COPE, Chief, Highway Statistics Section
and L. L. LISTON, Transportation Economist

Unit costs of highway construction and maintenance have almost doubled in the last 10 years, considerably exceeding the 77-percent increase in the basic cost of living. The price of gasoline, excluding tax, rose 62 percent in the same 10 years, but since the tax on gasoline rose less than 13 percent during this period the net effect felt by the motorist in his purchases of gasoline was an increase of only 47 percent in the total of price plus tax.

There has been a substantial increase in revenue for highways over the 10-year period, resulting from the tremendous growth in the number of motor vehicles. This has been nullified, however, by the extreme increase in highway costs and the contrastingly small one in tax rates. Current revenue will buy no more highway work than did the 1940 revenue. Yet the need is far greater, for the constantly multiplying number of vehicles continually intensifies our traffic problems.

The public apparently does not have a clear picture of the relation of taxes to the total cost of owning and operating an automobile. An analysis for a typical passenger car indicates that the taxes represent only 11 percent of the total ownership and operation costs—less than any other major item of cost involved. Put in practical terms, of the 6.6 cents per mile it costs to own and operate an automobile, all taxes combined represent seven-tenths of a cent.

The gasoline tax accounts for only 6½ percent of the total cost of owning and operating a car, or four-tenths of a cent per mile, and the tax rate is actually lower in proportion to individual income now than it was in 1940. Each cent of the gasoline tax rate costs the average motorist seven-hundredths of a cent per mile, or about 13 cents a week—just about 1 percent of the total ownership and operation cost.

GASOLINE TAXES constitute a very small part of the total cost of owning and operating an automobile, and revenues from gasoline taxes are falling far short of needed highway construction and maintenance expenditures. While other prices and costs have nearly doubled in the past 10 years, the tax on gasoline has increased only 12.7 percent. It is the purpose of this study to compare trends in gasoline taxes, costs of operating an automobile, and costs of constructing and maintaining highways.

A previous article in PUBLIC ROADS[1] reported a study of gasoline price and tax increases during a 30-month period ending June 30, 1948. Although there were substantial gasoline price increases in all States and gasoline tax increases in 12 States during that period, these increases had no measurable effect on consumption of gasoline. In the 1949 study, however, no effort was made to establish the relation between gasoline costs, total vehicle operating costs, and the portion of the vehicle operator's dollar that goes for road-user taxes. Such comparisons are made here.

Gasoline Prices and Taxes

Gasoline prices have continued to increase since 1948, and there have been increases in gasoline taxes in some States. The increases in gasoline prices (excluding taxes) and in State gasoline taxes during the 54-month period from January 1946 to June 1950 are shown in figure 1. This figure is an extension of figure 2 in the 1949 study, which covered the 30 months from January 1946 to June 1948. The findings for the longer period are in such complete agreement with those of the original study that reiteration of the conclusions in detail would be needless repetition.

The earlier study established that no gasoline tax increase has had a measurable effect on the consumption of gasoline, and none within reasonable contemplation appears likely to do so. Theoretically, any rise in the cost of operating automobiles should have some restraining effect on the amount of vehicle use. The reason that no effect has been noted is rather obvious—the gasoline tax constitutes only about 6.5 percent of the total cost of operating an automobile (as will be shown later), and has actually been declining in terms of its relation to the total cost.

There were net tax increases, during the 54-month period, of from 1/2 cent to 2 cents per gallon in 23 States, and a net tax decrease of 1 cent per gallon in one State. There were net price increases, excluding taxes, in all States during the same period. In Utah the price increase was only 1/2 cent per gallon, and in California the net price increase was only 2 cents per gallon. In other States, however, the net price increases were much higher—ranging from 3.4 cents per gallon in Arizona to 8.5 cents per gallon in Wyoming. Available information indicates that the Utah and California increases were relatively small because of intense competitive conditions in those areas.

Relative Cost of Gasoline

It has never been possible to measure the extent of the effect of operating costs on vehicle ownership and use. Much of the discussion and publicity on the point has centered around the cost of gasoline, probably because this is an item with which the public comes in daily contact. Gasoline taxes have assumed a disproportionate importance to the public because considerable publicity has been given to gasoline prices and taxes when matters pertaining to them have been under study by State legislatures, and to the allegation that "gasoline is cheap—only the tax is high."

The emphasis on gasoline taxes has tended to obscure the fact that many items of the cost of operating a motor vehicle considerably exceed gasoline tax payments. It is not generally recognized that both gasoline prices and gasoline tax rates have risen more slowly than general price levels. The average retail price of gasoline in 1940, including taxes, was 18.4 cents per gallon. During the 10 years to 1950 the retail price alone, excluding taxes, increased 62 percent. It is worth noting that this increase, though substantial, was considerably less than the 77-percent increase in the cost of living reflected in the Bureau of Labor Statistics

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[1] *The effect of tax increases on gasoline consumption*, by E. M. Cope and L. L. Liston. PUBLIC ROADS, Vol. 25, No. 7, March 1949, p. 138.

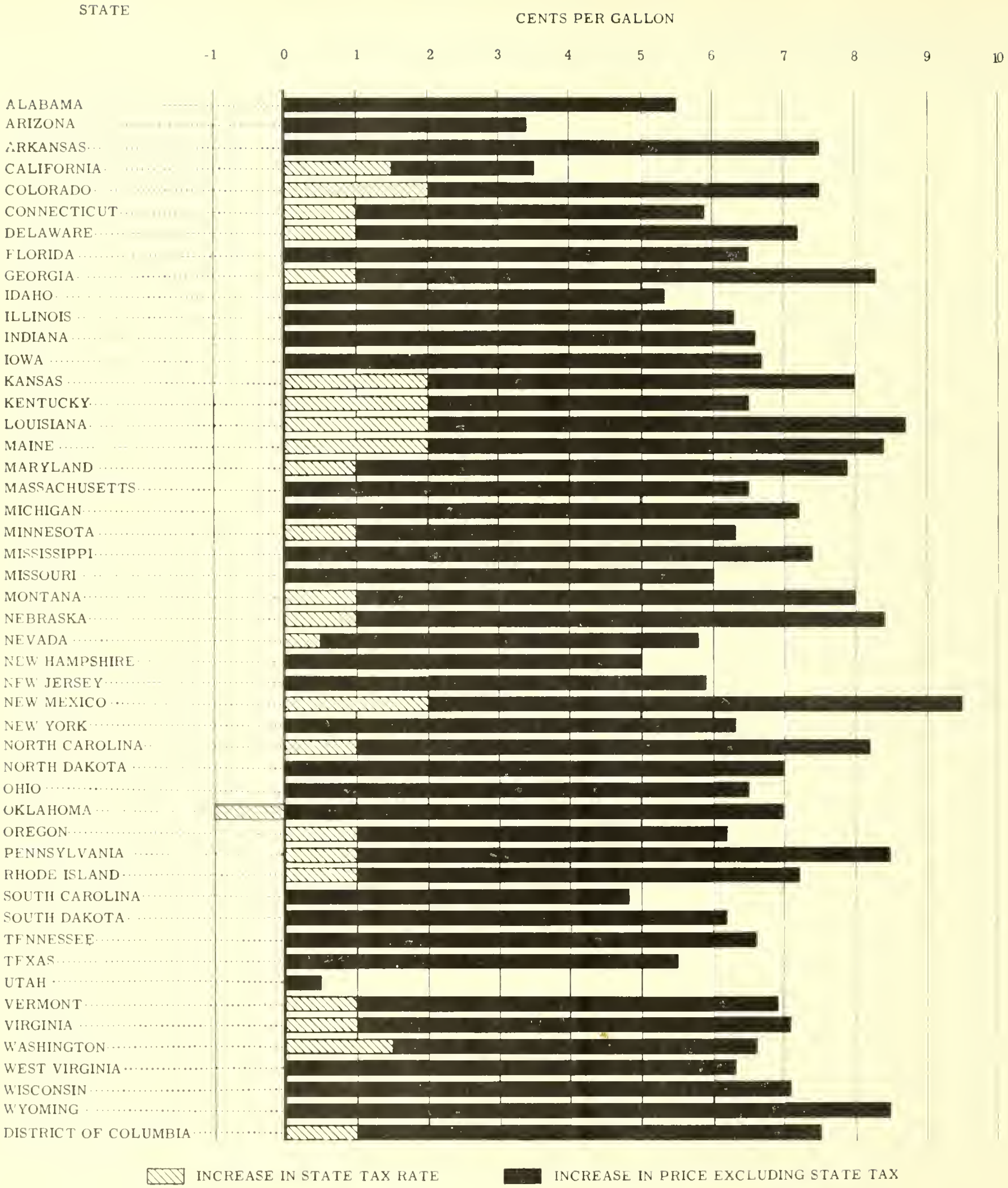

Figure 1.—Gasoline price and tax increases, January 1, 1946–June 30, 1950

index. In the same 10-year period the weighted average of State and Federal gasoline taxes rose only 12.7 percent. Thus, due to the relative stability of gasoline taxes, the total cost of gasoline to the consumer increased only 47.3 percent. Since wages have risen considerably more than the cost of gasoline, the consumer is actually in a better position now than in 1940 with respect to his purchases of gasoline. The percentage increase in the cost of gasoline, even though it was approximately five times as great as that for gasoline taxes, was still 20 percent less than the percentage increase in the cost of living reported by the Bureau of Labor Statistics. A comparison of gasoline taxes and prices in 1940 and in 1950 is given in table 1.

Highway Costs

The economic factors that brought about increased costs of gasoline also caused increased costs of raw materials, wages, transportation, and other components of the total price of motor vehicles, gasoline, and associated products. Likewise, they brought about sharp increases in the costs of constructing highways—a rise of 97 percent in the 10-year period from 1940 to 1950. The increases in the costs of construction are reflected in table 2.

During the same period highway maintenance costs rose 87 percent. This figure is a composite of the relative increases from 1940 to 1950 in the unit costs of the principal items of maintenance, which were as follows: Labor, 114.27 percent; material, 56.72 percent; equipment, 72.73 percent; and overhead, 67.10 percent.

Table 1.—*Gasoline price[1] and tax changes, 1940–50*

	Price, excluding tax	Tax	Total price	Relation of tax to total price
	Cents	*Cents*	*Cents*	*Percent*
1940	12.9	5.5	18.4	29.9
1950	20.9	6.2	27.1	22.9
Percentage change	+62.0	+12.7	+47.3	−7.0

[1] Prices in effect in the capital cities of the States except for Maryland and Oregon, where the prices are for Baltimore and Portland, respectively.

Table 2.—*Changes in unit costs of highway construction, 1940–50*

Item	1940	1950	Increase
			Percent
Common excavation: bid price, cu. yd	$0.21	$0.34	61.9
Concrete pavement: bid price, sq. yd	1.68	3.66	117.9
Structures:			
Reinforcing steel: bid price, lb	.045	.100	122.2
Structural steel: bid price, lb	.063	.139	120.6
Concrete: bid price, cu. yd	19.170	44.620	132.8
Composite index			97.0

Figure 2 shows the relative increases in the cost of living, the price of a new automobile, the price of gasoline (excluding tax), the gasoline tax rate, and the unit costs of highway construction and maintenance. It will be noted that the price of gasoline did not increase as much as either the cost of living or the price of a new automobile. But even the petroleum industry, generally considered to be one of the most efficient in our economy, found it necessary to increase the price of gasoline five times as much (on a percentage basis) as gasoline taxes were increased.

Automobile Operating Costs

State gasoline taxes, which are the principal source of revenue for highways, cost the average automobile user about 65 cents a week. This fact is of limited significance until it is related to total costs of vehicle operation. In order to establish the relation, an estimate of the cost of owning and operating an automobile is presented in table 3. It is not based on actual records for any particular vehicle or group of vehicles, but the figures are believed to be typical, at mid-1950 prices, for average operation of the type of automobile considered.

The estimate is based on a low-priced 1950-model four-door sedan built by a leading manufacturer, and covers an assumed 10-year life for the vehicle. The automobile is assumed to be registered in Baltimore, Md., and subject to normal taxes in that area. Although the mileages, operating costs, and other factors in the estimate are believed to be reasonable, they are not purported to be averages or for an average vehicle. Data for such purposes are not available in sufficient coverage. The essential factors of the estimate are stated in the notes below table 3.

In estimating expenditures for repairs and maintenance, the *Service Job Analysis* estimate compiled by Motor Service Magazine was used. This analysis lists the number of jobs of different kinds done by car dealers and independent repair shops for several years. A sample group of jobs, comprising more than two-thirds of the total listed, was chosen as representative of the maintenance and repair work which would be done on a typical automobile during the 10-year period. These items range from major repairs such as a complete engine overhaul to minor maintenance items like washing and lubrication. The frequency of the jobs listed in the *Service Job Analysis* and the experience of men familiar with automotive maintenance and service were used as guides to the number of times each job would be required during the life of the car. Costs, including parts, were then obtained from the most recent flat-rate manuals, and the prices of parts as listed in the manual were checked locally to determine any necessary adjustments for the Baltimore area.

The expenditures for major items—replacement tires, motor overhaul, painting, for example—are distributed over a period of years, rather than being charged entirely to the year of actual expenditure, because the benefit of each expenditure extends beyond the year in which it was made. The cost of the motor overhaul, for instance, occurs in the seventh year but is distributed over the seventh through the tenth years.

No costs for car financing, for fines and forfeitures, or for automobile club membership are included, nor is any interest on the investment included. Although these items may add substantially to the cost of owning and operating an automobile in some instances, the wide variance in their application among motorists makes it impractical to include them in this study.

Relation of Cost Elements

It is somewhat surprising to note that the several factors entering into the cost of operating the vehicle tend to keep the cost per mile within the rather narrow range of 7.35 cents in the first year to 5.24 cents[2] in the tenth year. The high depreciation and insurance costs of the first few years are offset to a considerable degree by greater mileage and low maintenance costs, and by absence of tire replacement expenditures. Although the estimate is based on the assumption that the vehicle will be scrapped after 10 years of use, the probability is, in a period in which there is a relative shortage of vehicles, that it would pass into the hands of a marginal user and continue in service. It is not likely, however, that the over-all cost per mile would

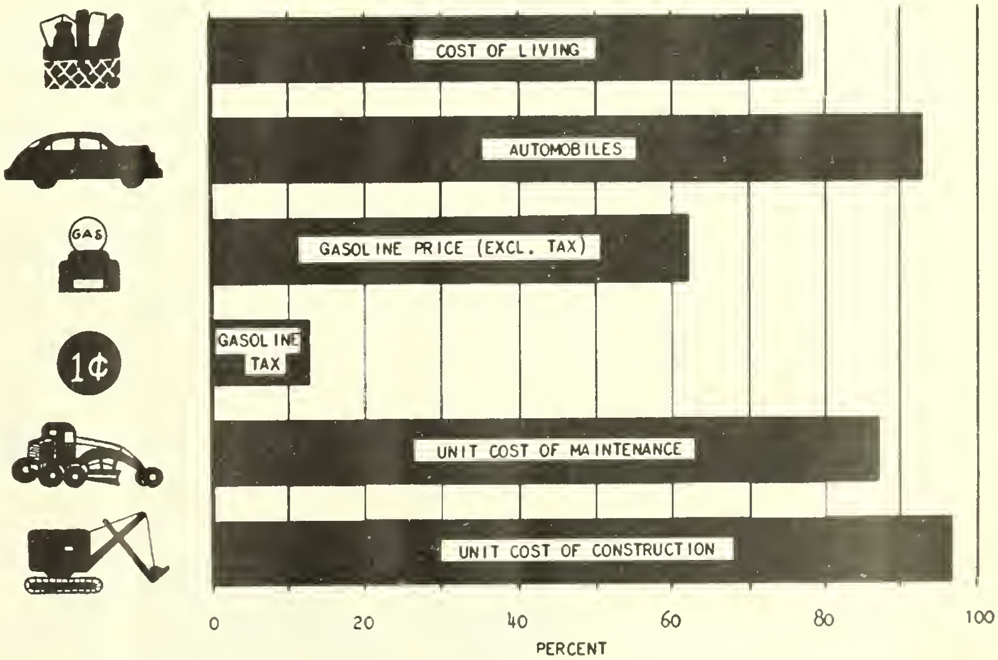

Figure 2.—Relative increases in unit costs of selected items, 1940–50.

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[2] These costs are all based on mid-1950 price levels. Practically all items of cost, with the exception of taxes, have increased since that time.

Table 3.—*Estimated cost of owning and operating an automobile*

Item	First year (12,500 miles)		Second year (12,000 miles)		Third year (11,000 miles)		Fourth year (10,000 miles)		Fifth year (10,000 miles)		Sixth year (9,500 miles)		Seventh year (9,500 miles)		Eighth year (9,000 miles)		Ninth year (8,500 miles)		Tenth year (8,000 miles)		Ten-year period (100,000 miles)	
	Total cost	Cost per mile	Total cost	Cost per mile	Total cost	Cost per mile	Total cost	Cost per mile	Total cost	Cost per mile	Total cost	Cost per mile	Total cost	Cost per mile	Total cost	Cost per mile	Total cost	Cost per mile	Total cost	Cost per mile	Total cost	Cost per mile
	Dollars	*Cents*	*Dollars*	*Cents*	*Dollars*	*Cents*	*Dollars*	*Cents*	*Dollars*	*Cents*	*Dollars*	*Cents*	*Dollars*	*Cents*	*Dollars*	*Cents*	*Dollars*	*Cents*	*Dollars*	*Cents*	*Dollars*	*Cents*
Costs excluding taxes:																						
Depreciation	360.00	2.880	330.00	2.750	200.00	1.818	125.00	1.250	100.00	1.000	75.00	0.790	75.00	0.790	65.00	0.722	40.00	0.471	30.00	0.375	1,400.00	1.400
Repair and maintenance	46.25	.370	57.74	.481	87.02	.791	112.04	1.120	113.30	1.133	173.49	1.826	159.18	1.676	129.09	1.434	70.36	.828	36.08	.451	984.55	.984
Replacement tires and tubes							26.00	.260	25.00	.250	25.00	.263	22.00	.231	22.00	.244	20.00	.235	20.00	.250	160.00	.160
Accessories	4.50	.036	14.84	.123	14.84	.134	14.84	.149	14.83	.148	14.83	.155	14.83	.155	14.83	.165	14.83	.174	14.83	.184	138.00	.138
Gasoline	163.46	1.307	156.80	1.307	143.67	1.307	130.73	1.307	130.73	1.307	124.07	1.307	124.07	1.307	117.60	1.307	111.13	1.307	104.47	1.307	1,306.73	1.307
Oil	16.94	.136	16.17	.135	16.94	.154	14.25	.143	15.68	.157	16.08	.170	12.71	.134	12.32	.137	9.69	.114	11.40	.143	142.18	.142
Insurance	116.00	.928	116.00	.967	116.00	1.055	111.00	1.110	111.00	1.110	59.00	.621	59.00	.621	59.00	.656	59.00	.694	59.00	.737	865.00	.865
Garaging, parking, tolls, etc	90.00	.720	90.00	.750	90.00	.818	90.00	.900	90.00	.900	90.00	.947	90.00	.947	90.00	1.000	90.00	1.059	90.00	1.125	900.00	.900
Total	797.15	6.377	781.55	6.513	668.47	6.077	623.86	6.239	600.54	6.005	577.47	6.079	556.79	5.861	509.84	5.665	415.01	4.882	365.78	4.572	5,896.46	5.896
Taxes and fees:																						
State:																						
Gasoline	41.70	0.333	40.00	0.333	36.65	0.333	33.35	0.333	33.35	0.333	31.65	0.333	31.65	0.333	30.00	0.333	28.35	0.333	26.65	0.333	333.35	0.333
Registration	10.00	.081	10.00	.083	10.00	.091	10.00	.100	10.00	.100	10.00	.105	10.00	.105	10.00	.111	10.00	.119	10.00	.125	100.00	.100
Titling	31.00	.248																			31.00	.031
Property	5.00	.040	5.00	.042	5.00	.045	5.00	.050	5.00	.050	5.00	.053	5.00	.053	5.00	.056	5.00	.059	5.00	.062	50.00	.050
Subtotal	87.70	.702	55.00	.458	51.65	.469	48.35	.483	48.35	.483	46.65	.491	46.65	.491	45.00	.500	43.35	.511	41.65	.520	514.35	.514
Federal:																						
Gasoline	12.51	.100	12.00	.100	11.00	.100	10.00	.100	10.00	.100	9.50	.100	9.50	.100	9.00	.100	8.51	.100	8.00	.100	100.02	.100
Oil	.66	.005	.63	.005	.66	.006	.75	.007	.83	.009	.72	.007	.50	.005	.48	.005	.51	.006	.60	.008	6.34	.007
Automobile, tires, parts, etc	21.00	.168	19.16	.160	11.73	.107	9.47	.095	9.11	.091	7.32	.077	6.76	.071	6.22	.069	4.02	.047	3.40	.043	98.19	.098
Subtotal	34.17	.273	31.79	.265	23.39	.213	20.22	.202	19.94	.200	17.54	.184	16.76	.176	15.70	.174	13.04	.153	12.00	.151	204.55	.205
Total	121.87	0.975	86.79	0.723	75.04	0.682	68.57	0.685	68.29	0.683	64.19	0.675	63.41	0.667	60.70	0.674	56.39	0.664	53.65	0.671	718.90	0.719
Total of all costs	919.02	7.352	868.34	7.236	743.51	6.759	692.43	6.924	668.83	6.688	641.66	6.754	620.20	6.528	570.54	6.339	471.40	5.546	419.43	5.243	6,615.36	6.615

Automobile.—The vehicle considered was a 1950-model four-door sedan, costing $1,450 excluding accessories and taxes. The junk value of the automobile at the end of 10 years was placed at $50.00.

Accessories.—Accessories provided included heater, three sets of seat covers, and side mirror.

Maintenance and repair.—Maintenance and repair covered routine lubrication, brake adjustment, replacement of minor parts such as spark plugs, ignition points, wiper blades, fan belts, etc. Additional items assumed were brake relining twice, clutch replacement twice, generator and starter overhaul once, universal joint replacement once, complete motor overhaul once, one new axle, one complete repaint job, kingpin and bushing replacement twice, and other intermediate and minor repairs. Fender and body work averaging about $8.50 per year was also included.

Replacement tires and tubes.—Purchase of 8 new tires and tubes during the life of the car was assumed.

Gasoline and oil.—Gasoline consumption was set at 15 miles per gallon; oil consumption at 422 quarts in the 10-year period, with cost and amount adjusted to the age and condition of the car.

Insurance.—Insurance coverage of $15,000/$30,000 public liability, $5,000 property damage, and comprehensive fire and theft, etc., was assumed for the full 10 years; $50-deductible collision insurance for the first 5 years.

Garaging, parking, tolls.—Monthly charges of $5.00 for garage rental or indirect cost of owner's garage and $2.50 for parking and toll fees were included.

Taxes.—Taxes represented are Federal and Maryland gasoline taxes at 1½ and 5 cents per gallon, respectively; Maryland registration fee of $15.00 per year, including the property-tax component; Maryland titling tax at 2 percent of retail price; Federal excise taxes on motor vehicles, tires, tubes, and accessories; and Federal tax of 6 cents per gallon on oil.

Benefit period.—The costs of certain major items are spread over a period of benefit rather than being charged entirely to the year in which the expenditure was actually made.

be appreciably reduced, provided the vehicle was maintained at a level to assure minimum safety and comfort.

Of the 7.35 cents-per-mile total operating costs in the first year of the estimate, the cost of gasoline, including all taxes, was 1.74 cents per mile. Gasoline taxes amounted to only 0.433 cent per mile, or 0.067 cent per mile for each cent of the tax rate. Thus each increment of 1 cent of tax adds less than 1 percent to the total cost of operating the vehicle. Under these circumstances it can be understood why no tax increase has ever made a measurable difference in the highway use of gasoline.

There is good reason to believe that the average motorist does not understand the true relation between his vehicle ownership and operation costs and the tax monies expended for the highways he uses. It would probably come as a surprise to him to learn that the total amount he pays in highway-user taxes, toward the construction and maintenance of the roads over which his automobile is operated, is substantially exceeded by all of the other major items of vehicle operation cost. This fact is presented graphically in figure 3.

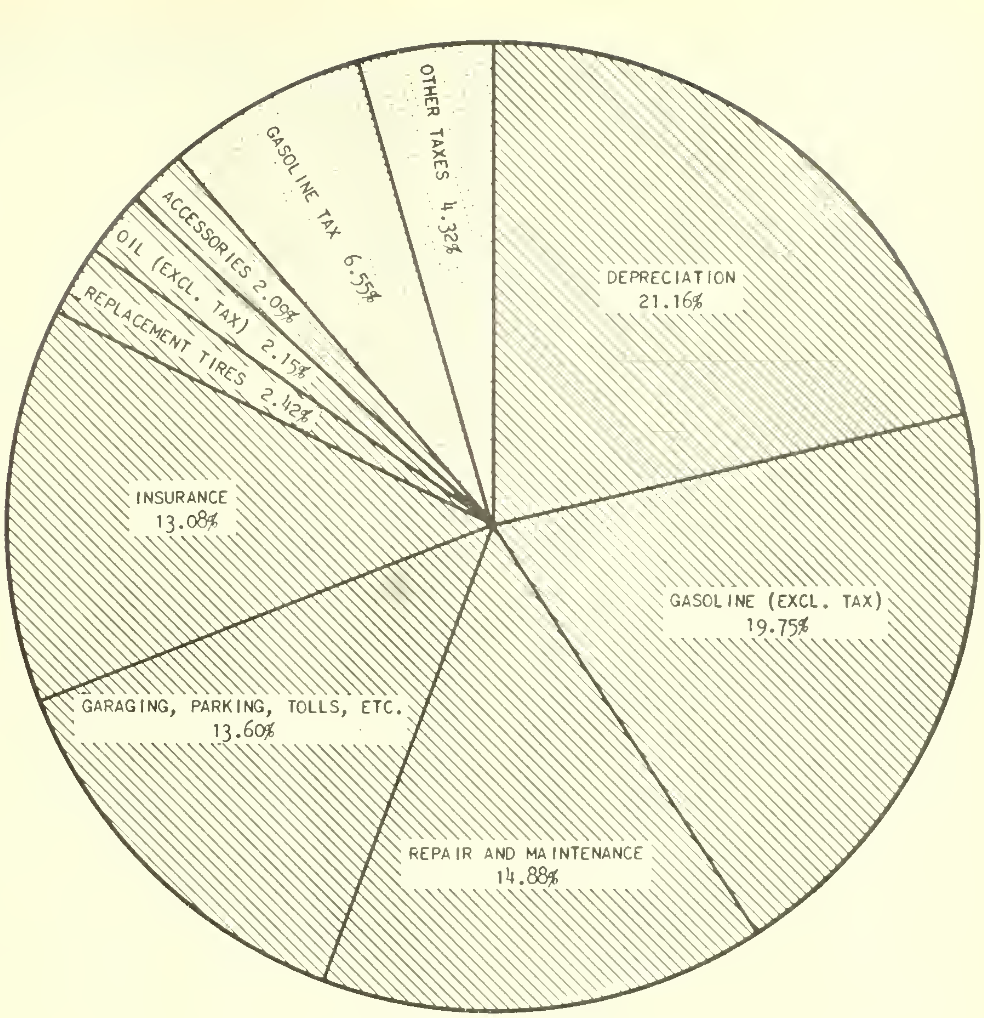

Figure 3.—Distribution of automobile ownership and operation costs.

Because of the widely quoted statements made in organized opposition to proposed increases in gasoline taxes in several States, most automobile owners might be reluctant to believe that each cent of the gasoline tax accounts for only 1 percent of total vehicle operating costs. The average motorist is also probably not aware that highway improvements made possible by each additional cent of gasoline tax not only add to the safety and convenience of motor-vehicle travel, but also reduce operating costs through improved surfaces, grades, and alinement, reduced mileage, and lessened traffic congestion.

Highway Revenues and Needs

Although road-user tax rates have risen less than any other automotive operating cost during the past 10 years, the revenues from them have increased substantially as a result of the large increase in the number of vehicles. The increases in highway construction and maintenance costs, however, have so reduced the value of the highway dollar that total revenues today will provide only about the same number of units of construction and maintenance that were purchased with 1940 revenues. While this reduction in the purchasing power of revenues is undoubtedly the most serious single problem now facing highway authorities, there are two others of almost equal importance—the mounting volume of traffic for which highway capacity must be provided, and the higher standards to which today's highways must be constructed to allow safe travel for present speeds and loads. If prices had remained at 1940 levels, current revenues would be adequate to maintain our then-existing system of roads and streets, and to improve and expand the system as needed for the greatly increased number of vehicles. But with present revenues able to purchase only the amount of construction and maintenance that was purchased with 1940 revenues, highway authorities are finding it impossible to provide adequate highway capacity for a 51-percent greater number of vehicles and a 60-percent increase in traffic volume.

Effect on the Individual

Discussions of road-user taxes are frequently of a technical nature not readily understood by the automobile owner. He is exposed to diametrically opposite views and irreconcilable statements. The allegation is made that "gasoline is cheap—only the tax is high." Yet the price of gasoline (excluding tax) now represents 19.8 percent of total automobile operating costs, while the gasoline tax is only 6.5 percent, and the tax portion of the total price of gasoline has decreased approximately 7 percent during the past decade. The motorist appears to be willing to pay tolls that cost the equivalent of an additional 15-cent tax per gallon for the use of controlled-access toll highways, but seems not to understand the relatively low cost, per vehicle-mile, of adequate free highways. When automotive taxes and highway programs are discussed, he unfortunately sees figures only in large multiples of any amounts familiar to him. Being impressed by their magnitude, he does not realize that the portion of these amounts that he pays as an individual is actually very small.

In most States there are certain minimum amounts that must be spent for highway administration, equipment, and maintenance. After these requirements have been met, any funds remaining are available for construction. Small increments in tax allocations under these circumstances can amount to relatively large increases in funds available for construction. If the 19 States having gasoline tax rates of less than 5 cents per gallon were to increase their rates 1 cent, the State funds available for highway construction would be increased, on the average, by more than 40 percent. In over two-thirds of these States the construction funds would be increased an average of more than 50 percent. Yet this increase would cost the motorist (at 15 miles per gallon), only 0.067 cent per mile, or about 13 cents a week.

Volume Changes in Sand-Gravel Concrete

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by F. H. JACKSON, Principal Engineer of Tests
and A. G. TIMMS, Senior Materials Engineer

Abnormal expansion, as evidenced in map-cracking, has been observed in concrete pavements built with sand-gravel aggregate in some midwestern States. Tests with this and two other aggregates of widely different mineral composition, combined with different types of cement and subjected to various weathering conditions, produced effects in the laboratory which correlated with those observed in the field.

Expansion of concrete specimens made with sand-gravel was much greater than that of similar specimens containing the other aggregates, and appeared to be due to some characteristic related to the mineral composition of the sand-gravel. The magnitude of the expansion was markedly influenced by the properties of the cements used.

The addition of crushed limestone to the sand-gravel, producing a normally graded coarse aggregate which contained about 50 per cent calcareous material, eliminated the abnormal expansion entirely.

Introduction

ABNORMAL EXPANSION of concrete pavement with resultant so-called "map-cracking" of the surface, which has been widely observed in certain midwestern states, particularly Kansas and Nebraska, prompted the Bureau of Public Roads about 10 years ago to initiate a series of tests to determine whether these effects could be reproduced in the laboratory and, if so, just what combinations of materials produced them and what steps might be taken to eliminate the trouble. The tests involved the measurement of changes in length of concrete specimens containing aggregates from three sources: The Platte River in Nebraska, Chicago, Ill., and Long Island, N. Y.

The Platte River aggregate was representative of a class of materials locally known as "sand-gravel," which contains only a small percentage of particles larger than 3/8 inch in size. Most of the concrete pavements in these areas which have shown distress were constructed with this type of aggregate. The aggregates from Long Island and the Chicago area were selected as representative of two sources differing widely in mineralogical composition but possessing excellent service records in concrete.

Each of the three aggregates, in three different gradations, was used in combination with each of four cements. All of the latter met the usual American Society for Testing Materials requirements for type I cement, and two of them in addition met both the A.S.T.M. requirements for type II cement and the requirements of the Board of Water Supply of New York City.

The concrete specimens made from these aggregates and cements were subjected successively to various types of laboratory weathering cycles which involved alternate wetting and drying together with alternate heating and cooling, but without freezing and thawing. In general the program followed the pattern set by W. E. Gibson, of the Kansas State Highway Laboratory, who began studying this problem as early as 1932. The results of Gibson's findings were reported before the Highway Research Board in 1938.[1]

A progress report giving the results of the studies made by the Bureau over a 2-year period was presented before the Highway Research Board in 1942.[2] This was followed in 1949 by a final report which reviewed the earlier work and gave the results of further measurements up to a total of 9 years of exposure.[3]

In the report presented here the data and discussions of the latter two papers have been combined into a single report which is intended to serve as a complete and final report of this investigation as it was originally planned.

Summary of Observations

The principal indications of these tests were as follows:

1. A laboratory weathering cycle which involved immersion in water at 70°F. for 24 hours followed by drying in air at 130°F. for 24 hours produced length changes and visual effects (such as map-cracking) in concrete test specimens that correlated with the behavior of similar concrete in the field.

2. Concrete specimens containing only the Platte River sand-gravel aggregate developed, in general, much larger expansions than similar concrete containing the other two aggregates. In many cases the appearance of map-cracks on the surfaces of the specimens provided visual evidence of abnormal expansion.

3. The abnormal expansion that developed when the Platte River sand-gravel was used as total aggregate appeared to be due to some characteristic related to its mineral composition rather than to its grading or to the high cement content normally used with this type of material.

4. In the concrete specimens containing the Platte River sand-gravel as total aggregate, the magnitude of the expansions was influenced to a marked degree by certain differences in the properties of the four cements used. However, there was no relation observed in this study between the amount of expansion and any of the physical or chemical properties of the cements.

5. Adding crushed limestone with a maximum size of 1½ inches to the Platte River

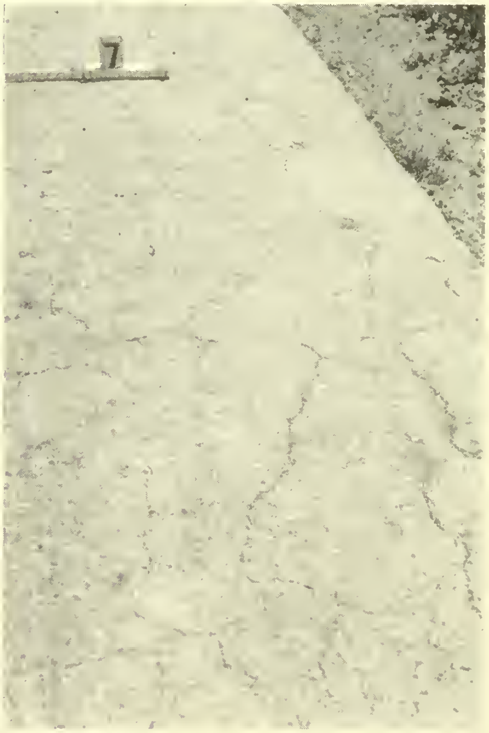

Figure 1.—Typical map-cracking of sand-gravel concrete pavement.

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[1] *A study of map cracking in sand-gravel pavements,* by W. E. Gibson. Proceedings of the Highway Research Board, Vol. 18, Part I, 1938, p. 227.

[2] *Volume changes in sand-gravel concrete,* by F. H. Jackson and W. F. Kellermann. Proceedings of the Highway Research Board, Vol. 22, 1942, p. 252.

[3] *Volume changes in sand-gravel concrete,* by F. H. Jackson and A. G. Timms. Proceedings of the Highway Research Board, Vol. 29, 1949, p. 212.

Table 1.—*Chemical and physical properties of portland cements*

		Portland cement			
		1	2	3	4
Chemical analyses (percent):					
Silica (SiO_2)		21.95	20.60	22.71	23.16
Alumina (Al_2O_3)		5.57	7.74	4.72	5.11
Ferric oxide (Fe_2O_3)		3.20	2.65	4.60	3.67
Lime (CaO)		62.70	62.70	62.54	64.28
Magnesia (MgO)		2.99	2.77	2.77	1.15
Sulfuric anhydride (SO_3)		1.70	1.79	1.51	1.32
Sodium oxide (Na_2O)		.26	.36	.19	.26
Potassium oxide (K_2O)		.67	.56	.43	.47
Alkali calculated as Na_2O		.70	.73	.47	.57
Water-soluble alkali calculated as Na_2O		.52	.38	.18	.21
Computed compound composition [1] (percent):					
Tricalcium silicate (C_3S)		42	38	39	42
Dicalcium silicate (C_2S)		32	31	36	35
Tricalcium aluminate (C_3A)		9	16	5	7
Tetracalcium alumino ferrite (C_4AF)		10	8	14	11
Calcium sulfate ($CaSO_4$)		2.8	3.0	2.5	2.1
Physical properties:					
Specific surface (Wagner)	cm.² per gm	1,850	1,705	1,965	1,815
Sugar test (Merriman):					
Neutral point	ml	33.2	36.8	2.7	2.7
Clear point	ml	18.1	57.5	2.7	2.7
Autoclave expansion	percent	.09	.42	.05	.01
Normal consistency	percent	24.5	25.5	23.5	24.0
Tensile strength (1:3 mortar):					
At 7 days	lb. per sq. in	305	340	315	360
At 28 days	lb. per sq. in	375	420	445	435

[1] The compound compositions given are in "shorthand" form.

sand-gravel, in the proportion of about 50 percent by weight of the total aggregate, eliminated the abnormal expansion. This was true for all four of the cements included in this investigation.

6. Concrete containing the sand and gravel from Long Island, an aggregate essentially siliceous in character, developed greater expansions than similar concrete containing the essentially calcareous sand and gravel from Chicago. This applied to all three gradings in which these materials were used and to all four cements.

7. Both the specimens containing a blend of the Platte River sand-gravel and crushed limestone and the specimens containing the Long Island sand and gravel, when used in normal gradation in combination with the two cements relatively high in computed tricalcium aluminate, developed greater expansions than similar specimens containing the two cements having relatively low percentages of this compound. However, this trend was not noted in the specimens containing the sand and gravel from Chicago.

8. These tests indicate that aggregate characteristics other than size and grading, such as particle shape, surface texture, mineral composition, etc., will affect the amount of cement necessary to maintain a given consistency, using a fixed water-cement ratio, to a much greater extent than has been commonly assumed.

Use of Sand-Gravel

The lack of suitable deposits of coarse aggregate for concrete in many parts of Kansas and Nebraska, and in certain sections of western Missouri and Iowa, has led to the extensive use of a naturally occurring mixture of sand and fine gravel as total aggregate for concrete work in these regions. This material, substantially all of which passes a ⅜-inch sieve, is known locally as "mixed aggregate" or "sand-gravel." Concrete in which it is used is known as sand-gravel concrete to distinguish it from concrete containing normally graded coarse aggregate, which is called locally "fine-and-coarse-aggregate" concrete.

The sand-gravels vary somewhat in mineral composition from place to place, but in general they are composed of quartz and granitic materials with varying amounts of feldspar and very little limestone or other calcareous material.

The local sand-gravel deposits, widely distributed along the beds of such streams as the Arkansas and Kaw Rivers in Kansas and the Platte River in Nebraska, furnish the only type of aggregate available in many parts of these States. These aggregates in general are reasonably well graded from ⅜-inch down, although the process of washing results in a deficiency in the finer sand sizes, particularly in the material passing the No. 50 sieve. The availability of these deposits makes them exceedingly attractive from the economic point of view, even though the fine grading has in some cases necessitated the use of cement contents as high as 7.5 to 8.0 sacks per cubic yard in order to meet requirements for design strength.

The belief that these materials were of satisfactory quality, and the fact that they were so readily available, resulted in their use in the construction of a substantial mileage of concrete pavements in Kansas and Nebraska. For example, most of the

Table 2.—*Grading, specific gravity, weight, and absorption of aggregates*

	Grading 1 (as used)	Grading 2 (as used)	Grading 3 [1]		
			Aggregate		Combined (as used)
			Fine	Coarse	
Grading: percentage retained on—					
1½-inch sieve				0	0
¾-inch sieve	0	0		55	36
⅜-inch sieve	5	5		83	54
No. 4 sieve	10	10	0	100	64
No. 8 sieve	30	30	20	100	72
No. 16 sieve	55	50	40	100	79
No. 30 sieve	80	65	60	100	86
No. 50 sieve	95	80	84	100	94
No. 100 sieve	100	95	97	100	99
Fineness modulus	3.75	3.35	3.01	7.38	5.84
Bulk specific gravity (dry):					
A—Platte River	2.61	2.61	2.61	[1] 2.60	
B—Long Island	2.65	2.65	2.66	2.63	
C—Chicago	2.62	2.62	2.62	2.65	
Weight, dry rodded (lb. per cu. ft.):					
A—Platte River	116	122	118	[1] 101	
B—Long Island	112	117	109	111	
C—Chicago	110	117	110	108	
Absorption (percent):					
A—Platte River	0.32	0.32	0.27	[1] 1.17	
B—Long Island	.29	.23	.31	.30	
C—Chicago	2.34	1.98	1.69	1.79	

[1] For this grading, where the Platte River combination was used, the materials include a combination of Platte River aggregate, grading 1, and sufficient crushed limestone to give desired grading.

Table 3.—*Other physical properties of aggregates*

		Aggregate		
		A—Platte River	B—Long Island	C—Chicago
Soundness (sodium sulfate loss at 5 cycles) [1]	percent	3.2	4.4	10.2
Resistance to abrasion (Los Angeles loss for grading D) [2]	percent	29.0	25.8	24.7
Organic matter (color test)		OK	OK	OK
Mortar strength ratio, 7 days [3]		.94	1.05	.90

[1] Weighted average loss based on grading 1.
[2] A.S.T.M. standard C 131-39, tentative revision 1942.
[3] A.S.T.M. standard C 87-42.

Table 4.—*Mineral analyses of aggregates*

Rock or mineral	Percentage composition for each sieve size indicated									Average composition[1]	
	1½- to ¾-inch	¾- to ⅜-inch	⅜-inch to No. 4	No. 4 to No. 8	No. 8 to No. 16	No. 16 to No. 30	No. 30 to No. 50	No. 50 to No. 100	Passing No. 100	Gravel	Sand
A—Platte River aggregate:											
Granite[2]	(3)	59	51	32	8	3	T(4)			55	7.1
Feldspar		19	25	40	34	21	11	4	3	22	18.8
Quartz		8	19	25	55	74	89	96	94	13.5	72.1
Chert		3	T	T	T	1	T	T	3	1.5	.6
Quartzite		4	3	1	3					3.5	.6
Sandstone		3	1	1	T	1				2	.3
Ferruginous sandstone		2								1	
Rhyolite		T	T	T	T					T	T
Epidosite		T								T	
Hornblende schist		T								T	
Andesite				T							T
Hornblendite					T						T
Biotite							T				T
B—Long Island aggregate:											
Quartz	87	71	74	83	92	93	95	96	98	77.3	92.8
Quartzite	13	16	9	5	4	3	T	1		12.6	2.2
Schistose quartzite		1	3	2						1.3	.3
Hornblende schist		1								.3	
Feldspar		3								1	
Ferruginous sandstone		4	3							2.3	
Mica schist		4	3	4	1					2.3	.8
Granite			6	4	1	3	T			2	1.3
Gneiss			2	2	1	1				.6	.7
Sandstone			T							T	
Garnet schist				T							T
Biotite							T				.3
Muscovite							T				T
Hornblende							T				.3
Sericite								T			T
C—Chicago aggregate:											
Dolomite	82	98	89	95	92	91	72	61	78	89.6	81.5
Quartzite	11		3				3	4		4.6	1.1
Trap	7		4	2	2	3	1	1	T	3.6	1.5
Limestone		2								.6	
Chert			4	2	4	2				1.3	1.3
Quartz				1	2	4	24	34	22		14.5

[1] Gravel size considered as material retained on No. 4 sieve, sand as that passing No. 4 sieve.
[2] Consists of quartz and feldspar with occasional biotite.
[3] This size Platte River aggregate not available.
[4] Trace.

original concrete pavement along U S 30 in Nebraska was of this type. This route follows the Platte River for many miles and the local sand-gravel is available at almost any point along the road with very short haul.

Unfortunately, the sand-gravel type of concrete pavement has not proved entirely satisfactory. Defects in the form of map-cracking of the surface frequently developed on many sections within a few years. Sometimes these defects led to progressive failure of a type which eventually required repair or replacement of the affected areas. A survey of Nebraska pavements conducted in 1939 by the Bureau of Public Roads in cooperation with the State Department of Roads and Irrigation revealed that about one-third of the approximately 500 miles of sand-gravel concrete pavement surveyed, all of which was constructed between 1925 and 1935, had developed map-cracking of a type which appeared to be progressive. In contrast to these observations on sand-gravel concrete, no evidence of map-cracking had developed up to 1939 on any of the pavements laid with fine-and-coarse aggregate concrete.

Map-Cracking

Map-cracking as used in this report may be defined as the type which forms a pattern of irregularly shaped blocks in the surface of the concrete. Map-cracking per se is not necessarily serious; nor does it necessarily lead to disintegration or complete failure. Some badly map-cracked pavements have proved durable under severe weathering conditions. However, when map-cracking is accompanied by other evidences of abnormal expansion, deep scaling, or unsoundness, as revealed by a lack of ring under the hammer, it can be generally assumed to be of the progressive type. An advanced stage of map-cracking of this type is shown in figure 1. Such a condition is evidence of disintegration even though under favorable conditions it may be possible to maintain traffic over the road for many years without the necessity of making extensive repairs or replacements.

The fact that map-cracking sometimes develops on roads carrying comparatively light traffic would indicate that the fundamental causes underlying the initial cracking are independent of this factor. The primary cause appears to be excessive and abnormal expansion of the concrete. Evidence of this is found in the closed expansion joints which usually accompany the appearance of map-cracking. Furthermore, this expansion seems to be confined to the sand-gravel concrete, being almost entirely absent on pavements containing the conventional fine-and-coarse aggregate type of concrete.

Object of the Investigation

One of the objects of this investigation was to determine whether the characteristic map-cracking which is frequently associated with the use of sand-gravel as total aggregate could be reproduced in the laboratory. As early as 1938 Gibson had shown that this type of failure could be developed in the laboratory by subjecting specimens of concrete to alternations of heating and cooling and wetting and drying without the introduction of a freezing cycle. It was considered desirable to continue the line of attack suggested by Gibson by making a series of tests which would include, in addition to the Platte River aggregate, materials from two other sources differing widely from it and from each other in mineral composition. It was also considered desirable to study the behavior of concrete containing these other materials when graded exactly the same as the Platte River aggregate. In addition, the tests were planned to compare the behavior of concrete containing the Platte River material with sufficient added crushed limestone to make a conventional total aggregate gradation, with that of concrete of the same proportions containing the other two aggregates. Complete descriptions of the materials, the mix data, and the weathering cycles used in the study follow.

Cements and Aggregates

Thirty-six combinations of materials involving four cements and three aggregates in each of three gradations were used in these tests. The cements were chosen to give a considerable range in chemical composition, particularly with respect to the percentage of computed tricalcium aluminate (C_3A). The results of physical tests and chemical analyses of the cements are given in table 1. Major differences in the cements were as follows:

Cement 1.—A.S.T.M. type 1, with 9 percent tricalcium aluminate (C_3A), low autoclave expansion, high Merriman sugar-test value, 0.70 percent total alkali, and 0.52 percent water-soluble alkali.

Cement 2.—A.S.T.M. type I, with 16 percent tricalcium aluminate (C_3A), relatively high autoclave expansion, high sugar-test value, 0.73 percent total alkali, and 0.38 percent water-soluble alkali.

Cement 3.—A.S.T.M. type II, with 5 percent tricalcium aluminate (C_3A), low sugar-test value, low autoclave expansion, 0.47 percent total alkali, and 0.18 percent water-soluble alkali. Cement 3, in addition to meeting the A.S.T.M. requirements, also met the requirements of the New York City Board of Water Supply. It would therefore be classified as a "Merriman" cement.[4]

Cement 4.—A.S.T.M. type II with 7 percent tricalcium aluminate (C_3A), low sugar-test value, low autoclave expansion, 0.57 percent total alkali, and 0.21 percent water-soluble alkali. Cement 4 was also classified as a Merriman cement.

It will be observed from table 1 that cement 2 has a considerably higher autoclave expansion than the other three. It is also the coarsest in terms of specific surface. It is of interest that cements 2 and 3 were from the same mill, the former being the regular commercial product and the latter a cement modified to meet the requirements of the New York Board of Water Supply. It should be noted that this work was initiated before the question of an alkali-aggregate reaction as possibly contributing to map-cracking had been raised. For this reason no attempt was made to secure wide variations in the alkali contents of the four cements.

The physical properties of the aggregates are given in tables 2 and 3 and the mineral composition of the various size fractions in table 4. All three of the aggregates met the conventional A.S.T.M. physical test requirements for concrete aggregates. A general description of the aggregates follows:

Aggregate A. — Sand-gravel from the Platte River at Schuyler, Nebr., composed essentially of granite, quartz, and feldspar, with about 0.3 percent material classified as opal. The amount of feldspar, predominantly potash (orthoclase and microcline), varied widely in the different sizes from a high of 40 percent in the No. 4–8 sieve size to a low of 3 percent in the material passing the No. 100 sieve. This aggregate has, in general, a poor service record.

Aggregate B.—Sand and gravel from Long Island, N. Y., (the so-called "Cowe Bay" material). The sand and gravel were almost entirely siliceous, being composed of about 90 percent quartz and quartzite, with practically no feldspar. This aggregate has an excellent service record.

Aggregate C.—Sand and gravel from Plainfield, Ill., in the Chicago area. In contrast with aggregate B, it was almost entirely dolomitic in composition, the sand being about 80 percent and the gravel about 90 percent dolomitic material. About 15 percent of the sand was quartz. This aggregate also has an excellent service record.

Table 5.—*Mix data*[1]

Cement and aggregate	Proportions by—		Actual cement content (sacks per cu. yd.)	Net water content (gal. per sack)	Weight of fresh concrete (lb. per cu. ft.)
	Dry weight	Absolute volume			
		Series I, Grading 1			
1–A	1:4.10	1:4.92	7.4	5.0	143
1–B	1:3.73	1:4.43	7.9	5.0	143
1–C	1:3.39	1:4.05	8.5	5.0	145
2–A	1:3.83	1:4.60	7.7	5.0	142
2–B	1:3.15	1:3.74	8.7	5.0	140
2–C	1:2.98	1:3.56	9.2	5.0	143
3–A	1:4.01	1:4.82	7.6	5.0	144
3–B	1:3.44	1:4.08	8.3	5.0	142
3–C	1:3.30	1:3.94	8.6	5.0	145
4–A	1:4.12	1:4.95	7.4	5.0	143
4–B	1:3.61	1:4.28	8.0	5.0	140
4–C	1:3.30	1:3.94	8.6	5.0	144
		Series I, Grading 2			
1–A	1:4.04	1:4.86	7.5	5.0	144
1–B	1:3.53	1:4.20	8.1	5.0	141
1–C	1:3.10	1:3.70	9.0	5.0	145
2–A	1:3.62	1:4.35	8.1	5.0	143
2–B	1:3.02	1:3.59	9.0	5.0	140
2–C	1:2.71	1:3.24	9.8	5.0	143
3 A	1:3.87	1:4.65	7.8	5.0	144
3–B	1:3.30	1:3.91	8.6	5.0	142
3–C	1:3.17	1:3.79	8.9	5.0	145
4–A	1:4.13	1:4.96	7.4	5.0	144
4–B	1:3.50	1:4.15	8.2	5.0	140
4 C	1:2.93	1:3.49	9.3	5.0	143
		Series II, Grading 1 [2]			
1–A	1:4.10	1:4.92	7.4	5.0	142
1–B	1:4.10	1:4.86	7.2	5.8	140
1–C	1:4.10	1:4.89	7.2	6.3	144
2–A	1:3.83	1:4.60	7.8	5.0	143
2–B	1:3.83	1:4.55	7.4	6.0	139
2–C	1:3.83	1:4.57	7.5	6.1	143
3–A	1:4.01	1:4.82	7.6	5.0	144
3–B	1:4.01	1:4.76	7.2	6.1	140
3–C	1:4.01	1:4.79	7.3	6.4	143
4–A	1:4.12	1:4.95	7.3	5.0	141
4–B	1:4.12	1:4.89	7.0	5.9	138
4 C	1:4.12	1:4.92	7.0	6.7	142
		Series II, Grading 2 [2]			
1–A	1:4.04	1:4.86	7.5	5.0	144
1–B	1:4.04	1:4.80	7.2	6.0	140
1–C	1:4.04	1:4.83	7.2	6.4	143
2–A	1:3.62	1:4.35	8.1	5.0	143
2–B	1:3.62	1:4.29	7.8	5.8	139
2–C	1:3.62	1:4.32	7.8	6.1	142
3–A	1:3.87	1:4.65	7.8	5.0	144
3–B	1:3.87	1:4.60	7.4	5.9	140
3–C	1:3.87	1:4.62	7.5	6.1	143
4–A	1:4.13	1:4.96	7.4	5.0	143
4–B	1:4.13	1:4.90	7.0	6.1	138
4–C	1:4.13	1:4.93	7.1	6.4	142
		Series I and II, Grading 3			
1–A	1:2.53:4.49	1:3.04:5.43	5.1	6.1	152
1–B	1:2.43:4.49	1:2.86:5.37	5.1	6.0	151
1–C	1:2.43:4.49	1:2.92:5.31	5.1	6.3	154
2–A	1:2.53:4.49	1:3.04:5.43	5.1	5.8	152
2–B	1:2.43:4.49	1:2.86.5.37	5.1	6.0	151
2–C	1:2.43:4.49	1:2.92:5.31	5.1	6.6	153
3–A	1:2.53:4.49	1:3.04:5.43	5.1	6.1	152
3–B	1:2.43:4.49	1:2.86:5.37	5.1	6.0	152
3–C	1:2.43:4.49	1:2.92:5.31	5.1	6.5	153
4–A	1:2.53:4.49	1:3.04:5.43	5.1	5.8	152
4–B	1:2.43:4.49	1:2.86:5.37	5.1	6.0	150
4–C	1:2.43:4.49	1:2.92:5.31	5.1	6.5	152

[1] Slump for gradings 1 and 2 in both series was approximately 1 inch; for grading 3, 2½ inches. Fineness modulus was 3.75 for grading 1, 3.35 for grading 2, and 5.84 for grading 3.

[2] In the case of aggregate A (Platte River) for Series II, the proportions and water content for each cement and grading are the same as was used for the corresponding combinations in series I. In the case of aggregates B (Long Island) and C (Chicago), the proportions were the same as were used for the corresponding combinations involving aggregate A. The water content was varied to maintain the desired slump.

Grading and Proportioning

Three different aggregate gradations were investigated. Sieve analyses of the various aggregates and aggregate combinations are

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[4] The term "Merriman" cement is frequently used to define portland cement that will meet the specifications of the Board of Water Supply of the City of New York, edition of 1937. These specifications, which contain certain special requirements not found in the A.S.T.M. specifications, were developed by the late Thaddeus Merriman, consulting engineer for the Board for many years. Mr. Merriman felt that his special requirements were necessary in order to insure proper burning, thereby insuring a more volume constant and a more durable product.

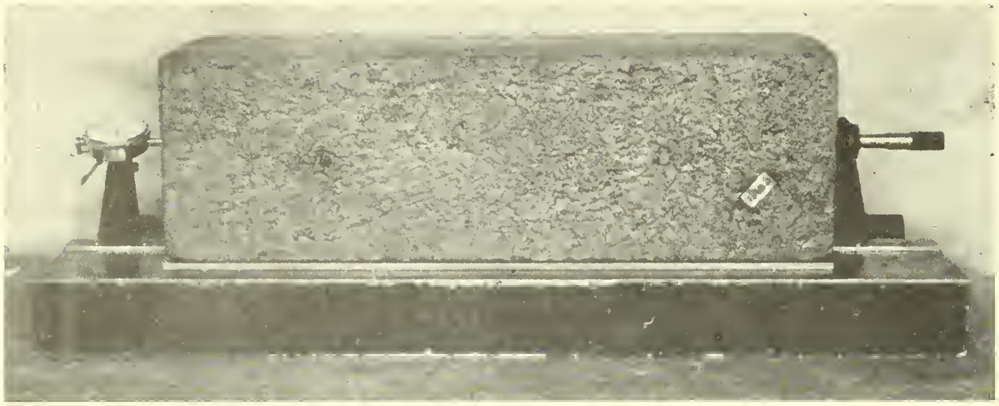

Figure 2.—Horizontal comparator with test specimen.

given in table 2. A summary of the gradings follows:

Grading 1.—The gradation of the Platte River sand-gravel as normally used: In reality a coarse sand, only 5 percent being retained on the ⅜-inch sieve and only 5 percent passing the No. 50 sieve.

Grading 2.—The same as grading 1 but with sufficient fines added (from the same source) to bring the total passing the No. 50 sieve up to 20 percent. The maximum size was not increased.

Grading 3.—A conventional aggregate gradation, from 1½-inch size down, with 64 percent retained on the No. 4 sieve. For aggregate A this was accomplished by adding crushed limestone (1½-inch to ⅜-inch in size) from Bethany Falls, Kans., to the Platte River material (a procedure locally known as "sweetening"), resulting in a combined aggregate consisting of 53 percent limestone and 47 percent sand-gravel by weight. For aggregates B and C the total aggregate was obtained by using gravel from the same source as the sand, the gradings being practically identical with that used for aggregate A combined with the crushed limestone.

Each of the four cements and three aggregates was tested in each of the three gradations, making 36 combinations in all. Two series of tests were run and two specimens were cast for each combination in each series, making a total of 144 specimens in the entire program.

In test series I, for gradings 1 and 2 the proportions were determined on the

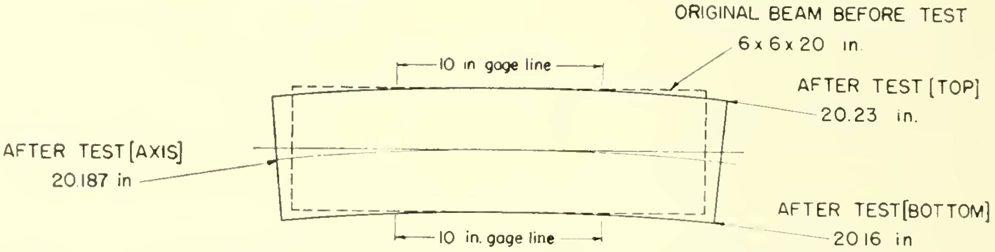

Figure 3.—Exaggerated sketch of test beam after various exposure cycles, showing typical expansions.

Table 6.—*Cumulative changes in length[1] after various storage periods*

Cement and aggregate	Test Series I: Percentage change in length after storage of—					Test Series II: Percentage change in length after storage of—				
	160 days (cycle A)	360 days (cycle B)	660 days (cycle C)	2,450 days (cycle D)	3,270 days (cycle E)	160 days (cycle A)	360 days (cycle B)	660 days (cycle C)	2,450 days (cycle D)	3,270 days (cycle E)
GRADING 1										
1-A	−0.005	0.001	0.772	0.940	0.968	−0.009	−0.002	0.752	0.952	0.976
1-B	−.005	−.002	.044	.108	.096	−.010	−.001	.102	.172	.169
1-C	−.017	−.010	.010	.038	.003	−.019	−.003	.016	.023	.001
2-A	−.005	.008	.118	.156	.143	−.010	.007	.124	.176	.147
2-B	−.008	.002	.038	.047	.034	−.011	.006	.022	.028	.017
2-C	−.019	−.006	.015	.052	.013	−.027	−.007	.011	.016	−.002
3-A	−.010	−.011	.223	.459	.508	−.009	−.005	.445	.648	.686
3-B	−.012	−.013	.010			−.017	−.005	.019	.032	.013
3-C	−.017	−.012	.006	.036	.004	−.023	−.003	.016	.023	.004
4-A	−.010	−.005	.316	.529	.548	−.015	−.010	.106		
4-B	−.016	−.011	.009	.043	.017	−.013	−.004	.017	.020	.004
4-C	−.022	−.015	−.002	.010	−.006	−.028	−.013	−.005	0	−.001
GRADING 2										
1-A	−0.010	0	0.498	0.605	.607	−0.010	−0.006	0.646	0.720	0.728
1-B	−.007	−0.004	.033	.072	.059	−.010	−.001	.041	.042	.057
1-C	−.018	−.015	.011	.026	0	−.019	−.011	.013	.019	0
2-A	−.006	.002	.082	.098	.093	−.015	.001	.115	.148	138
2-B	−.012	.003	.033	.060	.039	−.014	.002	.030	.039	.035
2-C	−.022	−.010	.018	.041	.010	−.021	−.006	.022	.036	.012
3-A	−.013	−.017	.030	.095	.099	−.020	−.019	.144	.204	.195
3-B	−.020	−.017	.012	.057	.045	−.017	−.006	.022	.033	.012
3-C	−.015	−.014	.010	.026	.009	−.021	−.008	.010	.013	.003
4-A	−.016	−.015	.008	.038	.011	−.015	−.014	.027	.041	.020
4-B	−.013	−.016	.007	.019	.008	−.019	−.010	.010	.019	.005
4-C	−.021	−.016	.003	.014	−.001	−.020	−.007	.008	.013	.001
GRADING 3										
1-A	−0.003	−0.001	0.022	0.042	0.033	−0.001	0.006	0.035	0.047	0.041
1-B	−.004	−.004	.062	.187	.163	−.006	.002	.101	.169	.184
1-C	−.004	.002	.009	.024	.016	−.012	.006	.019	.020	.013
2-A	−.004	0	.012	.107	.081	−.005	.003	.023	.108	.103
2-B	0	.004	.024	.072	.078	−.007	.004	.030	.068	.075
2-C	−.007	−.005	.007	.043	.014	−.018	−.003	.016	.029	.012
3-A	−.008	−.006	.005	.014	.009	−.007	.001	.016	.026	.018
3-B	−.010	−.003	.016	.030	.023	−.012	0	.028	.032	.032
3-C	−.013	−.001	.008	.027	.016	−.015	.001	.015	.017	.016
4-A	−.009	−.003	.004	.010	.007	−.009	−.002	.012	.015	.019
4-B	−.006	.001	.039	.046	.033	−.013	−.004	.014	.019	.010
4-C	−.009	−.001	.002	.013	.007	−.014	−.002	.014	.014	.011

[1] Each value is the average of tests of two beams.

basis of an approximately constant slump and a constant water-cement ratio of 0.67 by volume (5.0 gallons per sack), for all combinations of cements and aggregates. This resulted in average cement contents of about 7.6 sacks per cubic yard for the Platte River material, or about the same as used in the roads which had given trouble. The corresponding cement factors for aggregates B and C were considerably higher, averaging about 8.4 for aggregate B and 9.0 for aggregate C. For both gradings 1 and 2 a slump of approximately 1 inch was used, as this is the consistency normally employed in actual construction with sand-gravel aggregate.

In test series II, for gradings 1 and 2 identical proportions (by weight) and consistency were used for all three aggregates and a given cement as were used in series I with that same cement and the Platte River material (aggregate A). This was for the purpose of equalizing somewhat the variations in cement content which resulted from the use of a constant water-cement ratio in test series I. The resultant water-cement ratios in test series II varied considerably, running as high as 6.7 gallons per sack as compared to the 5.0 gallons per sack used in all mixes containing the Platte River material.

For grading 3 (the normal concrete grading) in both test series I and II, the combinations were proportioned on the basis of an approximately constant cement factor of 5 sacks per cubic yard, with the slump maintained at approximately 2½ inches.

It will be seen from the foregoing that, for each variable involving aggregate A in gradings 1 and 2, results based on four specimens of a kind (two in series I and two in series II) were obtained as compared to two specimens of a kind in the case of aggregates B and C. Thus, for aggregate A, series II served as a check on series I. For grading 3, all of the data from series II served as a check on series I inasmuch as the same mixes were used for both. Complete mix data are given in table 5.

Mixing and Storage

Aggregates in gradings 1 and 2 and the fine aggregate fraction of grading 3 contained some free water at the time of mixing. The coarse aggregates used in grading 3 were in a saturated and surface-dry condition at time of use. The necessary corrections for free water were made when computing the net water-cement ratios. Mixing was done in an open-pan Lancaster mixer, sufficient concrete for one 6 by 6 by 20-inch specimen being mixed at one time. One specimen for each of the three aggregates, the three gradings, and two of the four cements was made on each working day, making 18 specimens per day, or a total of 144 specimens in 8 working days. All specimens containing the ⅜-inch maximum size aggregate (gradings 1 and 2) were molded by thoroughly spading the concrete with a trowel, as it was found that the standard rodding procedure was not satisfactory for these mixtures. Specimens of normal concrete (grading 3) were molded by rodding in the standard manner. The consistency was not controlled directly by the use of the slump test but was judged by means of the flow test. The concrete mixes were adjusted to provide a consistency corresponding to approximately a 1-inch slump for gradings 1 and 2 and a 2½-inch slump for grading 3. All specimens were cured 1 day in the molds in the mixing room and 27 days in the moist room, prior to testing.

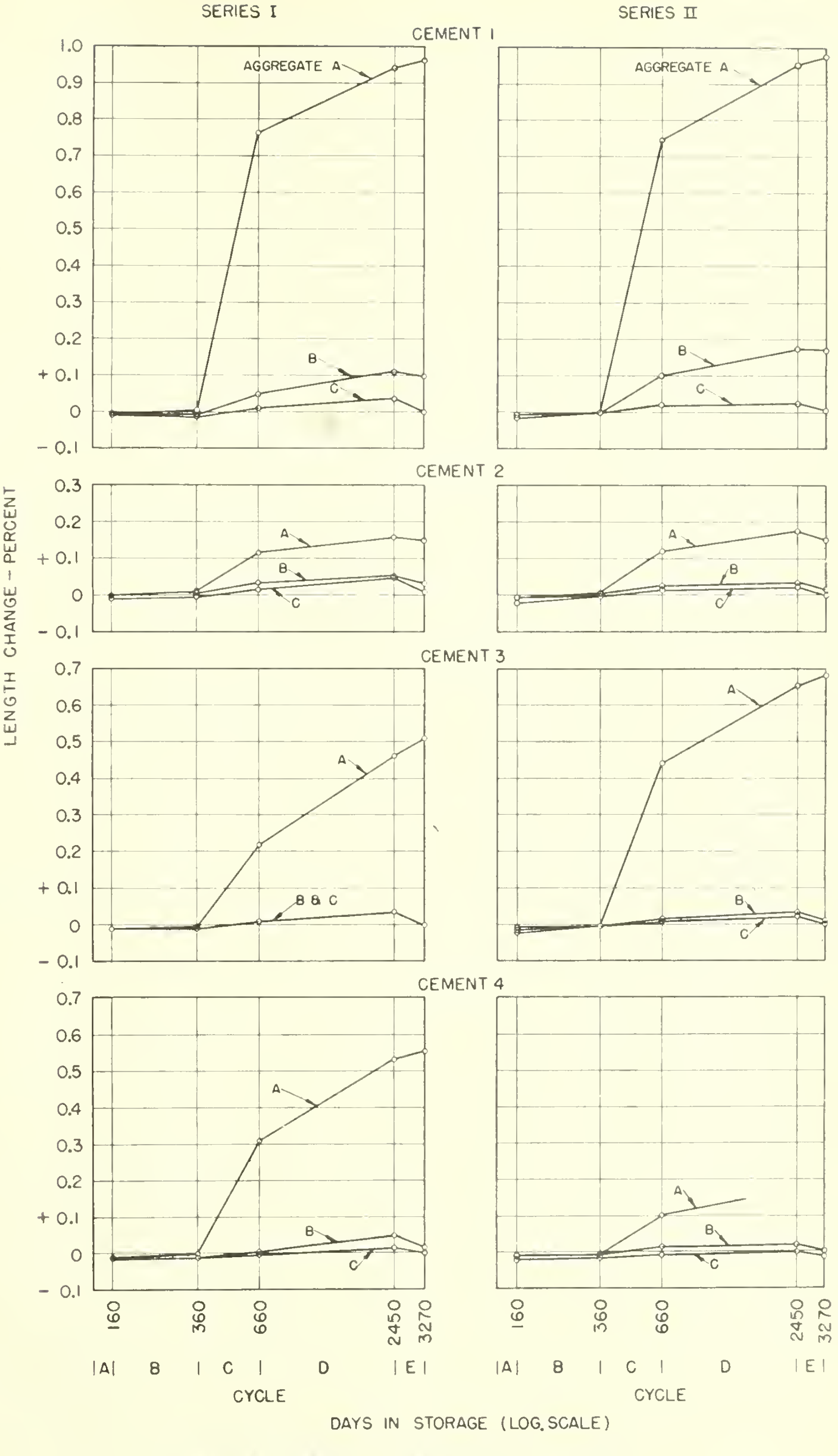

Figure 4.—Effect of aggregate type: Grading 1.

After a preliminary moist storage period of 28 days, the concrete test beams were measured for length and then exposed successively to a series of cycles of heating and cooling and wetting and drying. As will be seen later, the pattern of cycles was established during the course of the work, as need for change in procedure was evidenced. The complete series of cycles was as follows:

Cycle A.—One day in moist room at 70° F., followed by 1 day in air at 130° F., followed by 2 days in air at 70° F.: 40 cycles, requiring 160 days.

Cycle B.—One day in water at 70° F., followed by 1 day in air at 130° F., followed by 2 days in air at 70° F.: 50 cycles, requiring 200 days.

Cycle C.—One day in water at 70° F., followed by 1 day in air at 130° F.: 150 cycles, requiring 300 days.

Cycle D.—Continuous storage in moist air at 70° F. for 1,790 days (approximately 5 years).

Cycle E.—Exposure outdoors near Washington, D. C., for 820 days (approximately 2¼ years), followed by 4 days resaturation in water.

Measurements

The test specimens used in this study were concrete beams 6 by 6 by 20 inches in size. Stainless steel plugs were set in the center of each end of the specimens and brass inserts, provided with drilled gage seats, were set in the upper and lower surfaces. Three sets of length-change readings were taken—one on the end plugs and the others along the upper and lower surfaces of the specimens. These latter measurements were made with a mechanical strain gage over a 10-inch gage length; the end measurements were made with a horizontal comparator reading to 0.0001-inch. The comparator is shown in figure 2. All measurements were taken with the concrete in a moist condition at 70° F. in order to eliminate insofar as possible differences due to variations in temperature and moisture conditions at the time of test.

In general the surface measurements showed the same trends as those taken along the central axis. However, the surface measurements, which were made with a mechanical strain gage over a gage length of 10 inches, were somewhat erratic. The end measurements were made with a horizontal comparator over the entire 20-inch length of the beam and were quite consistent. The individual discrepancies noted in the surface readings reveal the difficulty of securing consistent results with a mechanical strain gage, which involves the personal equation. With the horizontal comparator, on the other hand, very consistent results are obtainable because the personal equation is absent.

Discussion of Observations

The surface measurements, though erratic, do indicate two definite trends, neither of which would have been revealed by end measurements. During cycle D (continuous moist storage for 1,790 days) the surface measurements revealed appreciable warping in many of the specimens, particularly those which had developed large residual expansions at the end of cycle C. These were principally the combinations involving aggregate A (the Platte River material) in gradings 1 and 2. The over-all expansion of these specimens at the end of cycle D was also large. In other words, they continued to expand after the wetting and drying cycle had been discontinued.

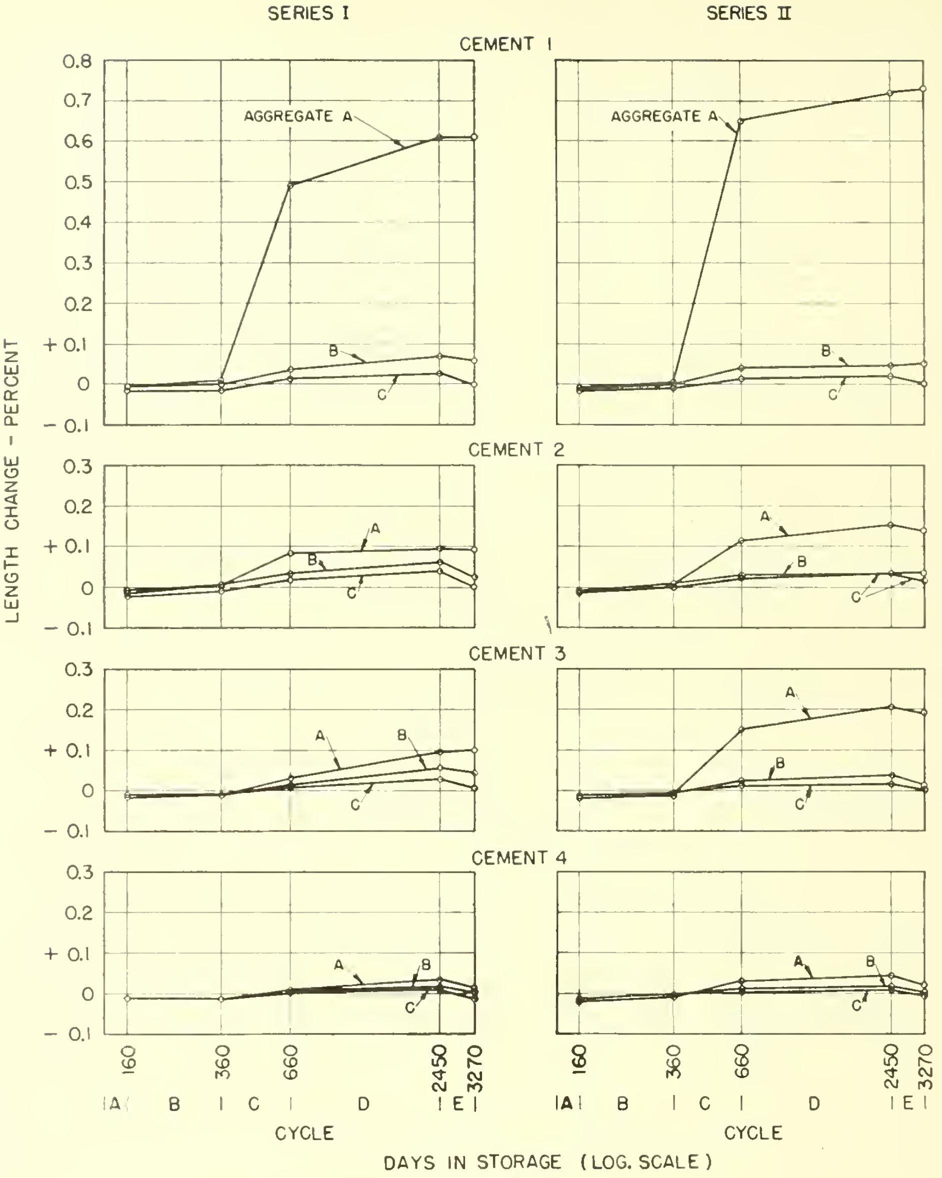

Figure 5.—Effect of aggregate type: Grading 2.

Warping was revealed by substantially higher expansion along the top surface than along the lower surface. This caused the ends to curl downward, as illustrated in figure 3, which shows readings for one of the beams in the combination of aggregate A (in grading 1) with cement 1. These measurements no doubt reflected the tendency of the surface mortar to expand at a greater rate than the mass of the concrete. Such a tendency probably exists in most concrete pavements because, due to finishing operations, a layer of mortar of distinctly inferior quality is formed on the surface of the pavement. These length differentials between the upper and lower surfaces were found only in the combinations which had expanded excessively. Where the over-all expansions were small (0.1 percent or less), which was the case with the majority of the combinations, the warping tendency was not indicated by the measurements. This was due, it is believed, to the fact that the surface measurements were not sufficiently precise to reveal the very small length changes involved.

Another interesting trend was the upward warping at the ends of most of the specimens at the end of cycle E (outdoor storage on the ground for 2¼ years). This warping was probably due to differential drying out of the top surface with respect to the lower surface, since the latter was in contact with the ground. Here again, however, the individual results show many inconsistencies. Although the general trends indicated by the surface measurements are probably bona fide, it is believed the individual measurements, for the reasons stated, are not sufficiently accurate to permit detailed comparisons between combinations where the total movements are small.

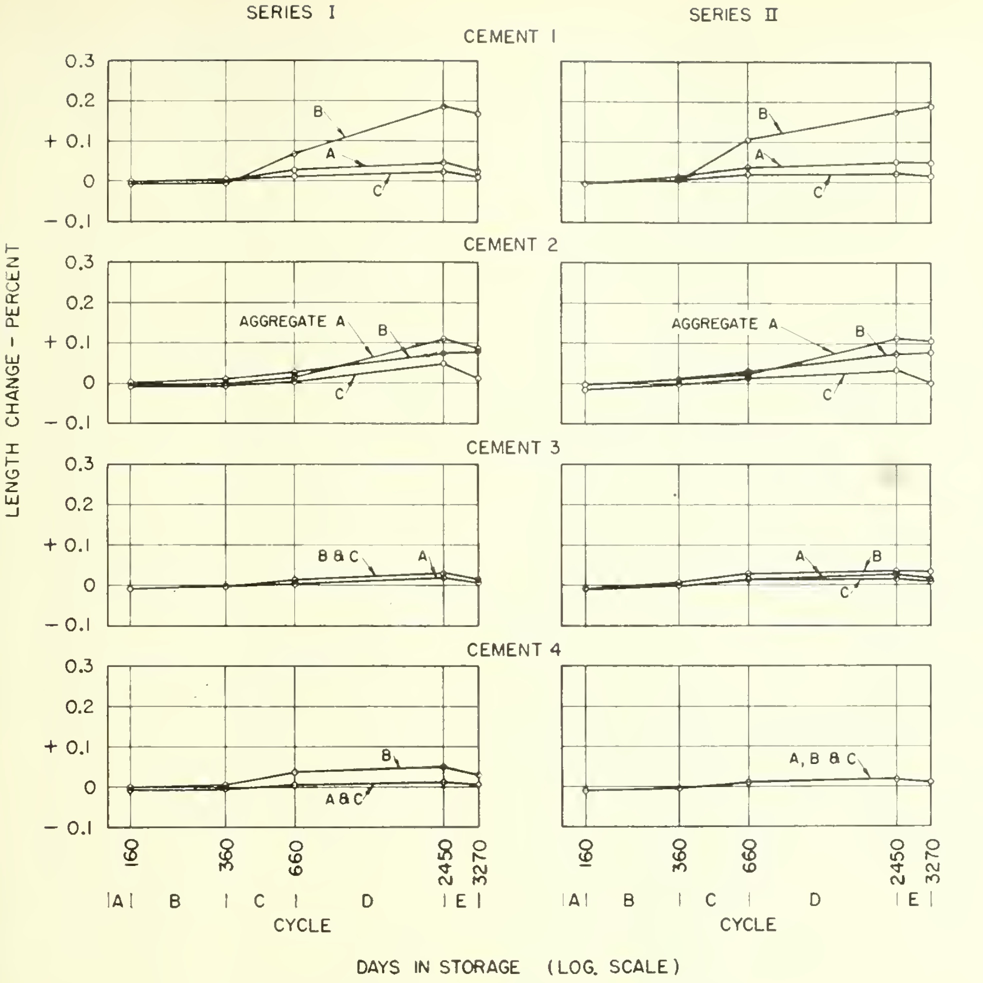

Figure 6.—Effect of aggregate type: Grading 3.

These include most of the combinations involving aggregates B and C as well as aggregate A in grading 3. For this reason these data are omitted and the balance of the report will be limited to a discussion of the end measurements only.

The cumulative percentage changes in length of the test specimens along the central axes, at the expiration of the various exposure periods, are given in table 6 and are plotted in figures 4–7. Each value is the average of measurements on two specimens made on different days. In the case of aggregate A in gradings 1 and 2, and all three aggregates in grading 3, the same proportions and consistency were used in both test series I and II. In this case, therefore, the results of the two series are directly comparable, a point which should be borne in mind when studying the data. On the other hand, combinations involving aggregates B and C in gradings 1 and 2 are not strictly comparable in the two series due to the fact that a constant water-cement ratio was used in series I whereas in series II the weight proportions were kept constant for each cement. Even in these cases, however, reference to the data will show that the differences in cement content and water content, although quite large, were apparently not sufficiently great to affect the general trends to an appreciable extent.

At the end of cycle A (160 days), it was found that all of the specimens were showing small residual contractions. This indicated quite definitely that 24 hours in the moist room was not long enough to insure complete resaturation after drying at 130° F. Cycle B was therefore introduced, to provide for immersion in water instead of storage in the moist room. After 50 alternations of cycle B, however, the net expansions were still found in all cases to be very small.

Cycle C was then instituted, omitting altogether the 48-hour storage period in air at 70°F. Under this procedure the specimens were immersed immediately in water upon removal from the drying oven at 130° F., thus simulating the effect of a sudden cool shower upon a concrete pavement at the close of a hot dry day. This treatment had the effect desired: It differentiated quite definitely between the concretes in a manner similar to that observed in service.

Cycle C was continued for 300 days. Abnormal expansion (an increase in length of almost 1 percent in certain cases) had developed in some of the Platte River sand-gravel combinations by that time, with resultant map-cracking and other evidences of deterioration. This indicated the desirability of discontinuing the type of cycle which involved daily handling of the specimens. They were therefore stored in the moist room for approximately 5 years (cycle D), after which they were again measured for length and then stored outdoors on the ground in the vicinity of the laboratory (cycle E). The final set of readings was taken after about 2¼ years of storage outdoors, at which time the specimens were approximately 9 years old.

Effect of Aggregate in Grading 1

Figure 4 shows for grading 1 and for each cement the effect of aggregate type on length change. It will be seen that the various combinations involving aggregate A, all of which showed relatively high expansion at the end of cycle C, continued to expand through cycle D and, in most cases, through cycle E. The concrete containing cement 2 is the only exception to the latter tendency. On the other hand, the concretes containing aggregates B and C, although showing small additional expansions at the end of cycle D, all contracted as the result of the outdoor exposure of cycle E. These facts would indicate that whatever the nature of the internal reactions which caused the excessive expansion with aggregate A, the same reactions were continuing through cycle E. Otherwise, contractions would be expected during cycle E similar to those shown in the case of aggregates B and C.

It should also be noted that the amount of the expansion occurring in the concrete containing aggregate A was influenced appreciably by the cement, cement 1 being by far the most active and cement 2 the least active of the four. When studying figure 4, it is important to remember that, insofar as aggregate A is concerned, the proportions used in series II were identical with those used in series I, although an interval of nearly a month elapsed between the casting of the specimens in the two series. The similarity of the trends as revealed in the two series is quite striking and tends to show that the trends shown in series I were not accidental.

It should be observed, also, that the influence of aggregate grading has been entirely eliminated by this grouping of the data since all of the specimens, regardless of aggregate type, contain material of identical grading (in this case grading 1, the normal grading of the Platte River material). The data show conclusively that aggregate grading is not responsible for the excessive expansions which have taken place.

As noted above, all of the concretes containing aggregates B and C expanded somewhat during cycle D and then showed contraction at the end of cycle E. Furthermore, aggregate B, the siliceous material from Long Island, consistently showed higher expansions than aggregate C, the dolomitic material from Chicago. There is a strong possibility that these differences, although small as compared to the expansions found in the case of aggregate A, may be related to differences in the thermal

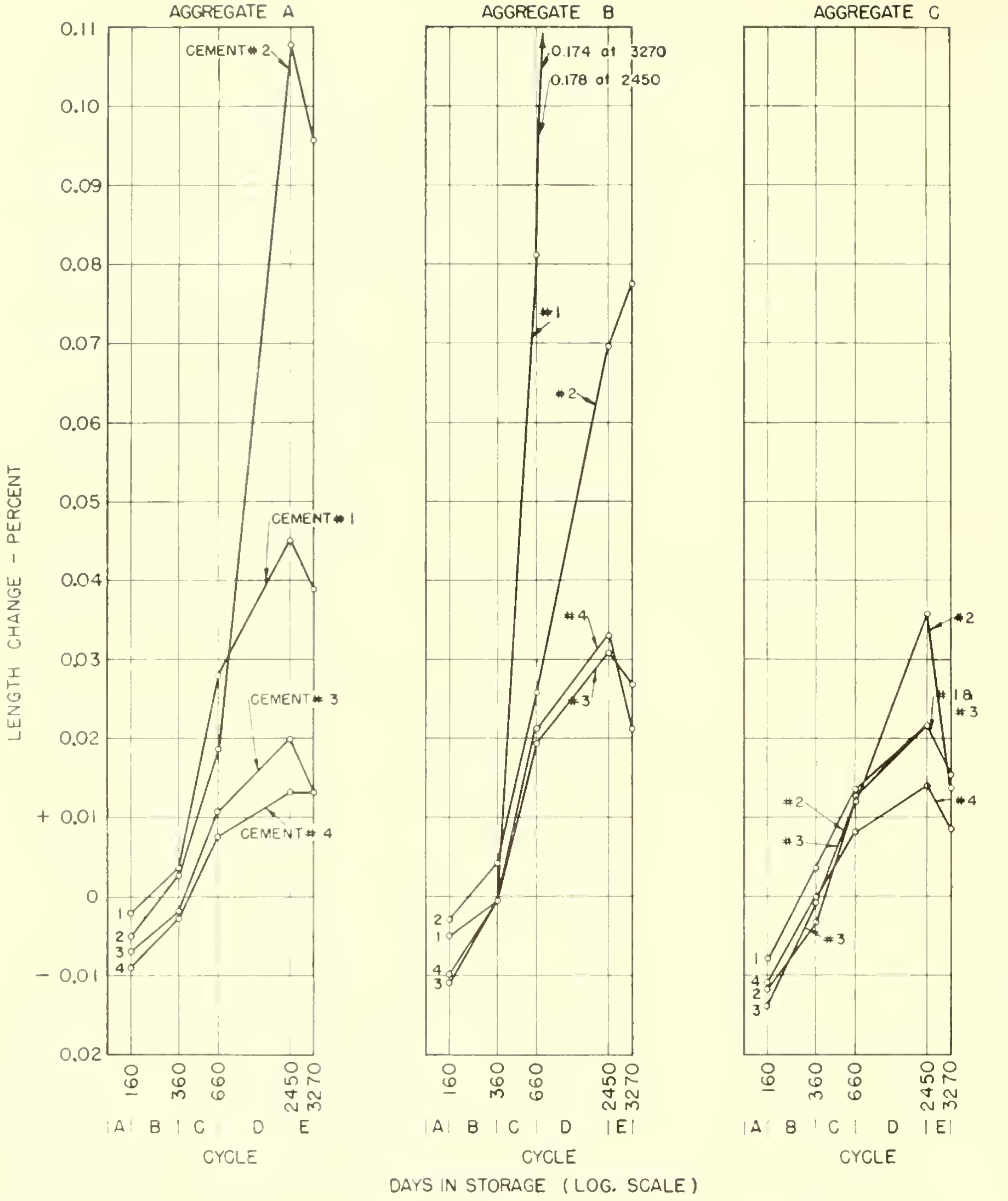

Figure 7.—Effect of cement: Grading 3 (average for tests series I and II).

characteristics of the two aggregates. However, there is no evidence of abnormal growth with either of these materials. The maximum expansions with all but one cement were less than the 0.1 percent, which is frequently indicated as the dividing line between normal and abnormal behavior in this regard.

The reason for the abnormal expansions found with the Platte River material remains unknown. There is the possibility of a slight alkali-aggregate reaction, but the fact that the Platte River aggregate used in these tests contained only 0.3 percent opal, coupled with the lack of relation between the alkali contents of the four cements and the resultant volume change, would tend to minimize this possibility. Neither can these abnormal expansions be due to the rich mix that was used, because in series I the cement contents of the combinations involving aggregates B and C were actually higher than those which involved aggregate A.

There remains the possibility that these abnormal effects are due to differential thermal characteristics of the aggregates. Blanks[5] has called attention to the fact that aggregate of the same general nature as the Platte River material contains substantial amounts of the orthoclase and microcline feldspars and that these minerals have a very low thermal coefficient of expansion. The Platte River gravel used in these tests contained varying amounts of these minerals, ranging up to as much as 40 percent in the Nos. 4–8 sieve size. It is possible that the deterioration of concrete containing this material may be due, at least in part, to stress developed within the concrete as the result of thermal incompatibility. This fact alone, however, would not account for the wide differences in expansion noted with the different cements. These differences must be related to some characteristic or characteristics of the combination which are influenced by both the aggregate and the cement. Whether these effects are physical or chemical, or a combination of the two, has not been determined.

Effect of Aggregate in Grading 2

Figure 5 shows, for each cement, the effect of aggregate type when used in grading 2. This grading was the same as grading 1 except that sufficient fine material from the same source was added to increase the total amount passing the No. 50 sieve from 5 percent to 20 percent, with no increase in the amount retained on the ⅜-inch sieve.

A comparison of figures 4 and 5 indicates, for cements 1 and 2, trends almost identical in grading 2 with those found with grading 1. In the case of cement 1, the addition of fines, while causing some reduction, did not appreciably decrease the excessive expansion shown by aggregate A in grading 1. In the case of cement 2, about the same expansions were noted for both gradings, the values being comparatively low in both cases. In the case of cement 3 in series I, and cement 4 in both series, the abnormal expansion shown for aggregate A in grading 1 was largely eliminated. In fact, so far as cement 4 is concerned, the expansions found for all three aggregates were all well within the 0.1-percent limit previously mentioned. In other words, for this cement, as well as for cement 3 in series I, concrete containing the Platte River material in grading 2 behaved normally.

In general it may be said that the addition of fines to the Platte River material reduced the expansions somewhat, the amount of the reduction varying with the cement. Moreover, the tendency for continued expansion with cements 1, 3, and 4 during cycle E, noted in the case of grading 1, was not found with grading 2.

Attention has been called to the fact that, in series I, the cement contents of the concrete containing aggregates B and C were substantially higher than those used with aggregate A. For example, in the case of aggregate C in grading 2, cement 2 of series I (fig. 5), the cement factor was 9.8 sacks per cubic yard as compared to 9.0 for aggregate B and 8.1 for aggregate A. These values, as well as the corresponding cement factors for the other combinations used in series I, gradings 1 or 2, are shown in table 5. The variations were necessary in order to maintain the same water-cement ratio throughout the series. The data show definitely that there is no relation whatever between cement content and expansion. In fact, as will be seen later, the actual expansions of certain combinations in gradings 1 and 2, all of which involved the use of very rich mixes in series I, were no higher at any time than the expansions developed with grading 3 (the normal concrete), where the cement content was reduced to 5 sacks per cubic yard.

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[5] *Modern concepts applied to concrete aggregate*, by R. F. Blanks. Proceedings of the American Society of Civil Engineers, Vol. 75, No. 4, April 1949, p. 441.

Effect of Aggregate in Grading 3

The effect of aggregate type in grading 3 with each cement is shown in figure 6. It will be noted that in practically all cases the abnormal expansions which occurred when using aggregate A in gradings 1 and 2 were eliminated by the use of a normal concrete grading obtained by mixing the Platte River material with crushed limestone. In fact, the maximum expansions with this combination were actually less in all cases, except with cement 2, than were those for the corresponding combinations involving aggregate B. In studying figure 6 it should be borne in mind that aggregate A, the Platte River sand-gravel, has been combined with crushed limestone in about equal proportions. This substantially reduced the silica content of the aggregate, which may account for the smaller expansions found in the case of aggregate A as compared to aggregate B, which was almost entirely siliceous. In the case of cement 2, both series show higher expansions for aggregate A than for either B or C. This reversal cannot be considered accidental since the results of the two series check each other almost exactly.

Effect of Cement in Grading 3

Except for the combination involving cement 1 and aggregate B, virtually all of the expansions observed in the normal concrete (grading 3) were within the range of 0 to 0.1 percent. It was decided, therefore, to plot the length changes for this grading on a very much larger vertical scale than that used in figures 4–6. In figure 7, the chart so produced, differences in expansion of the order of 0.01 percent or less are clearly indicated and may be of some interest. Each point used in plotting the curves was the average of four determinations—two in series I and two in series II. The corresponding length changes for each series separately are shown in table 6.

In studying figure 7, the nature of the various exposure cycles should be kept clearly in mind. It will be recalled that cycle A involved a 72-hour drying period followed by 24 hours in moist air. Figure 7 shows clearly that 24 hours of resaturation in moist air was not sufficient to prevent residual contraction. Furthermore, there seemed to be a definite tendency for cements 3 and 4 to show greater contractions than cements 1 and 2 (except the aggregate C–cement 2 combination), this trend being evident in the case of all three aggregates in both series. Also, for a given cement, concrete containing aggregate C showed somewhat greater contraction than the concretes in which aggregates A and B were used. At the conclusion of cycle A the moist storage condition was changed to 24 hours in water instead of in moist air, the period of drying remaining the same. At the expiration of 50 cycles of this treatment (cycle B), all of the specimens had expanded, although in most cases they still showed some residual contraction.

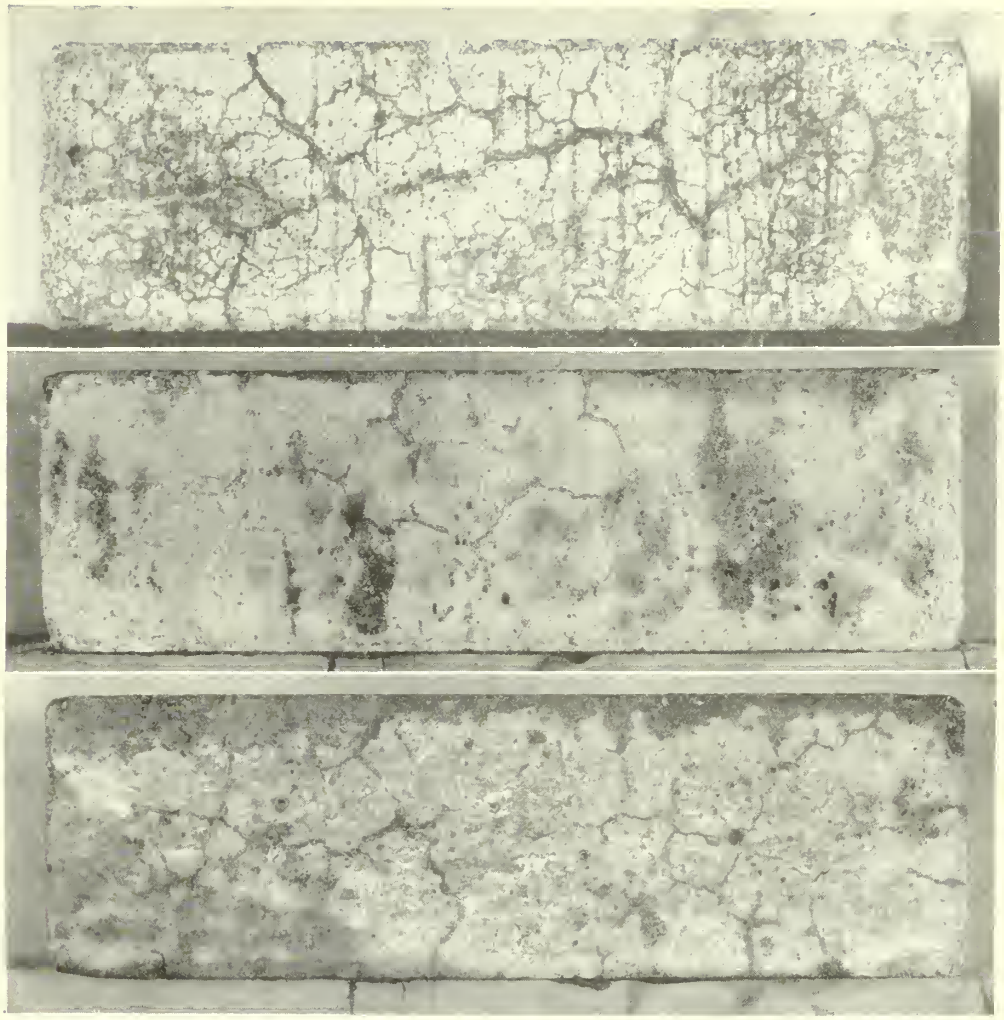

Figure 8.—Abnormal expansion of specimens containing sand-gravel aggregate: (top) grading 1 with cement 1, at end of cycle C; (center) grading 2 with cement 1, at end of cycle E; (bottom) grading 1 with cement 3, at end of cycle E.

With the elimination of the 48-hour drying period at 70°F. (cycle C), the specimens continued to expand, this trend continuing through cycle D. It will be noted that at the expiration of cycle C all combinations showed residual expansion, the amounts varying with both the cement and the aggregate. Cements 1 and 2 expanded more than cements 3 and 4, the difference being much more marked in the case of aggregates A and B than in the case of aggregrate C.

In most cases the specimens contracted during cycle E. After 2¼ years of outdoor exposure, the 96 hours in water which was provided at the end of cycle E was apparently not sufficient to resaturate the specimens. In the case of cement 2 with aggregate B, readings at the end of cycle E showed further expansion, which is a definite exception to the general trend. However, it should be noted again that the same tendency was found in both series (see fig. 6).

The chief point of interest observable in figure 7 is the relatively high expansion which took place in the combinations involving cements 1 and 2 with aggregates A and B. Aggregate A in this grading was composed of a mixture of the Platte River material and crushed limestone in about equal parts; aggregate B was almost 100 percent quartz or quartzite; and aggregate C about 95 percent dolomite. The chief difference in the cements is the fact that cements 3 and 4, in addition to meeting the standard A.S.T.M. requirements, also met the specification requirements of the New York City Board of Water Supply.

Cement meeting these requirements would be classified as A.S.T.M. type II, principally because of the limitation on alumina, but they are much more restrictive than type II in that they also include limitations on sugar solubility (a test devised by Merriman to detect underburning) as well as a requirement on maximum allowable water-soluble alkali. The authors are not prepared to advance an explanation for the increase in expansion noted in the case of cements 1 and 2 in combination with aggregates A and B as compared to the expansion noted for other combinations. However, the trends are so definite and are repeated so consistently in the two series of tests as to make it extremely unlikely that they are accidental. The data are presented as an interesting example of the possible variations in the volume change characteristics of different combinations of cements and aggregates. They illustrate a principle which is being recognized more and more: The volume constancy of concrete is influenced to a marked degree by the particular combination of cement and aggregates used in the mix, and these materials must be studied in combination with each other in concrete rather than individually.

Examples of the excessive map-cracking which developed in many of the specimens containing aggregate A are shown in fig. 8.

Unsolved Problems

It is realized that the foregoing discussions raise many questions for which answers have not been supplied. This is just as true now as it was in 1942 when the original progress report was prepared. For example, no one, as far as the authors are aware, has yet advanced an entirely satisfactory explanation for the abnormal expansion which takes place when Platte River or similar aggregate is used in concrete with certain cements. Nor has an adequate explanation been forthcoming as to why this expansion can be stopped by the addition of crushed limestone to such aggregates.

These tests, as well as tests made by other investigators, have proved that conventional factors such as aggregate grading, aggregate quality as measured by conventional tests, cement content, free lime in cement, etc., are not sufficient to explain this abnormal expansion. The probability that the aggregate is mildly alkali-reactive, combined with the fact that from one-fourth to one-third of the aggregate consists of a type of feldspar having a low thermal coefficient of expansion, may supply the answer, but the data of these tests indicate quite definitely that neither of these facts taken separately is the answer. Moreover, the fact that the Platte River aggregate used in these tests contained only 0.3 percent opal, and insignificant amounts of other possibly reactive aggregates, would indicate that the alkali-aggregate reaction, as it is generally considered, was not an important factor.

It is also difficult to visualize the mechanics of an action which results in expansion due to the use of aggregate having a low thermal coefficient of expansion. However, there is also the possibility that the feldspar may contribute to the expansion in some other way, due possibly to alteration, with time, of its physical properties.

New Publication

A FACTUAL DISCUSSION OF MOTOR-TRUCK OPERATION, REGULATION, AND TAXATION

The Bureau of Public Roads has recently published *A Factual Discussion of Motor-truck Operation, Regulation, and Taxation*, a 113-page bulletin presenting in summary form the factual records and other data available to the Bureau, with discussions of their significance, which might prove useful in any study and investigation of the transportation problem. The bulletin may be purchased from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 30 cents a copy.

The subject material in the bulletin is presented in seven parts, covering the growth of motor-vehicle registration and use, the effects of size and weight of vehicles on the geometric design and traffic capacity of highways, axle loading and its effect on roads and legal limitation, weight of vehicles and its effect on bridges, the character of overloaded vehicles and their pay loads, highway-user tax payments in relation to highway revenues and expenditures, and the allocation of highway tax responsibility. Several appendixes containing valuable information are also included.

The report was prepared at the request of the Subcommittee on Domestic Land and Water Transportation of the Committee on Interstate and Foreign Commerce, United States Senate, which was investigating problems relating to the transportation and communications industries. The report was presented before the committee in June 1950 and is reprinted from the committee's hearings, which appeared under the title *Study of Domestic Land and Water Transportation*.

Highway Soil Engineering Film

Highway Soil Engineering, a motion picture produced by the Bureau of Public Roads to illustrate field surveying and sampling and laboratory testing of soils encountered in highway construction, is now available on loan to highway departments, universities, and other organizations. The 16-millimeter film, with sound and in full color, has a running time of almost two hours. The subject treatment is technical in nature and of interest primarily to engineers and engineering students.

Highway Soil Engineering describes the two distinctive components of soil—granular and silt-clay materials—and illustrates the methods employed in surveying and sampling soils in the field. Laboratory tests are presented in sequences which show the step-by-step procedures involved. The tests demonstrated are those used by the Bureau and many of the State highway departments, and cover the complete range needed to examine the properties of soils that are of interest to highway engineers.

The picture shows the striking contrast in condition and in maintenance requirements of pavements laid on good and on poor subgrade soils. It concludes with a demonstration of the value of a sand subbase in preventing intrusion of subgrade soil into the overlying crushed-stone base course.

Highway Soil Engineering may be borrowed by any responsible organization without cost, except for the nominal transportation charges. The film may be obtained for showings by writing to the Visual Education Branch, Bureau of Public Roads, Washington 25, D. C.

U.S. GOVERNMENT PRINTING OFFICE: 1951—911331

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1940, 10 cents. 1942, 10 cents. 1948, 20 cents.
1941, 15 cents. 1946, 20 cents. 1949, 25 cents.
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Annual Report, Bureau of Public Roads, 1950. 25 cents.

HOUSE DOCUMENT NO. 462

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1943. 1944. 1945.

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Bibliography on Highway Lighting.
Bibliography on Highway Safety.
Bibliography on Land Acquisition for Public Roads.
Bibliography on Roadside Control.
Express Highways in the United States: a Bibliography.
Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF APRIL 30, 1951

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$17,376	$10,660	$5,459	329.9	$5,660	$2,810	85.8	$16,652	$8,081	405.2	$32,972	$16,350	820.9
Arizona	281	7,603	5,383	182.0	327	225	5.0	5,271	3,756	78.1	13,201	9,364	265.1
Arkansas	6,835	8,467	4,561	259.2	3,247	1,834	96.7	14,139	7,002	384.5	25,853	13,397	740.4
California	4,399	50,070	13,420	229.2	9,623	4,832	56.3	61,251	29,233	299.8	120,944	47,485	585.3
Colorado	3,595	4,798	2,712	179.2	3,165	1,604	59.4	12,673	7,054	279.1	20,636	11,370	517.7
Connecticut	3,519	6,206	3,135	23.0	2,029	1,209	2.3	6,504	3,619	8.9	14,739	7,963	34.2
Delaware	2,273	1,004	599	22.5	10	9		5,893	2,880	38.2	6,907	3,488	60.7
Florida	6,762	6,918	3,588	124.6	8,574	4,378	148.9	18,991	9,354	393.2	34,483	17,320	666.7
Georgia	6,774	11,642	5,959	155.0	11,446	4,829	116.8	29,809	14,941	675.4	52,897	25,729	947.2
Idaho	5,591	8,195	5,179	258.3	1,213	741	114.5	7,780	4,384	207.5	17,188	10,304	580.3
Illinois	18,654	39,770	21,414	347.2	31,803	15,897	400.5	50,478	26,216	311.0	122,051	63,527	1,058.7
Indiana	13,147	36,118	18,057	177.1	4,352	1,985	29.5	18,402	9,527	99.6	58,872	29,569	306.2
Iowa	2,083	13,655	6,370	621.0	11,608	5,746	514.3	13,155	6,948	309.7	38,418	19,064	1,445.0
Kansas	6,146	8,669	4,164	1,071.0	7,667	3,843	331.0	9,970	4,967	502.8	26,306	12,974	1,904.8
Kentucky	1,599	22,878	11,711	331.5	2,369	1,187	53.5	19,690	9,733	320.2	44,937	22,631	705.2
Louisiana	6,010	14,388	6,842	108.8	10,459	4,972	65.8	18,865	10,026	217.7	43,712	21,840	392.3
Maine	4,265	5,210	2,764	65.6	924	503	3.5	7,440	3,927	72.4	13,574	7,194	141.5
Maryland	5,049	4,886	2,420	19.4	3,026	1,228	15.8	12,006	5,533	41.6	19,918	9,181	76.8
Massachusetts	2,139	16,574	5,874	28.7	5,750	2,672	.3	65,663	32,528	61.9	87,987	41,074	90.9
Michigan	9,939	17,832	8,831	485.5	8,906	4,553	209.3	44,856	19,110	315.9	71,594	32,494	1,010.7
Minnesota	3,725	9,248	4,989	1,122.1	12,900	6,901	772.7	17,989	9,521	282.5	40,137	21,411	2,177.3
Mississippi	8,057	7,705	4,044	234.9	7,755	4,100	208.3	10,047	5,005	307.0	25,507	13,149	750.2
Missouri	10,393	22,791	11,712	785.8	16,692	8,475	441.4	30,126	15,919	434.7	69,609	36,106	1,661.9
Montana	6,051	14,883	8,978	564.6	3,549	2,064	61.2	13,051	7,862	268.8	31,483	18,904	894.6
Nebraska	7,785	15,300	7,993	532.8	7,240	3,586	243.1	11,818	6,217	330.1	34,358	17,796	1,106.0
Nevada	5,790	2,289	1,890	33.0	335	282	33.3	3,600	2,973	143.3	6,224	5,145	209.6
New Hampshire	2,540	2,921	1,634	25.3	959	478	8.5	4,907	2,438	42.8	8,787	4,550	76.6
New Jersey	5,058	7,598	3,799	12.5	3,341	1,670	.5	16,956	7,924	25.0	27,895	13,393	38.0
New Mexico	4,633	1,811	1,159	42.1	1,300	838	43.3	11,240	7,175	284.5	14,351	9,172	369.9
New York	46,180	49,264	26,266	133.2	16,540	7,587	20.5	109,999	51,569	241.4	175,803	85,422	395.1
North Carolina	5,303	14,467	7,168	419.7	4,321	2,095	86.3	28,495	13,766	621.1	47,283	23,029	1,127.1
North Dakota	3,362	10,963	5,613	1,762.4	5,827	2,889	449.6	5,266	2,631	439.2	22,056	11,133	2,651.2
Ohio	15,962	21,787	10,250	227.8	9,302	5,463	172.5	77,612	38,655	302.2	108,701	54,368	702.5
Oklahoma	1,462	18,543	8,683	305.9	6,899	3,746	159.2	24,435	12,298	401.4	49,877	24,727	866.5
Oregon	433	3,768	2,227	91.6	7,563	3,758	102.8	14,300	8,148	186.1	25,631	14,133	380.5
Pennsylvania	13,194	20,926	10,735	37.8	5,775	2,886	7.9	88,022	43,400	237.9	114,723	57,021	283.6
Rhode Island	1,635	5,388	2,694	43.2	1,532	766	3.0	13,900	7,068	16.0	20,820	10,528	62.2
South Carolina	2,983	11,131	5,988	190.1	2,306	1,143	44.5	9,652	5,001	264.5	23,089	12,132	499.1
South Dakota	1,177	10,719	6,242	898.9	5,041	2,768	324.0	9,573	5,891	668.4	25,333	14,901	1,891.3
Tennessee	3,988	11,481	5,615	160.2	12,102	5,825	197.9	19,793	9,198	351.4	43,376	20,638	709.5
Texas	8,482	4,771	1,796	83.9	23,197	11,673	372.5	50,460	24,057	927.0	78,428	37,526	1,383.4
Utah	2,095	5,952	4,461	144.2	1,243	924	22.9	4,661	3,397	164.1	11,856	8,782	331.2
Vermont	1,002	4,273	2,258	62.6	879	443	8.4	4,551	2,235	29.0	9,703	4,936	100.0
Virginia	5,840	24,496	12,091	657.3	7,372	3,683	199.0	15,312	7,482	283.2	47,180	23,256	1,139.5
Washington	5,173	8,196	3,058	166.7	1,919	951	44.3	20,899	10,001	116.9	31,014	14,010	327.9
West Virginia	4,406	13,172	5,225	89.3	5,467	2,738	71.7	9,237	4,662	76.9	27,876	12,625	237.9
Wisconsin	6,735	22,505	12,722	619.0	12,404	5,808	223.1	14,320	7,148	332.4	49,229	25,678	1,174.5
Wyoming	434	2,790	1,813	102.6	1,641	1,069	31.1	8,932	5,596	248.1	13,363	8,478	381.8
Hawaii	1,908	6,884	2,922	13.5	1,294	447	.5	10,584	3,913	29.9	18,762	7,282	43.9
District of Columbia	2,787	5,912	2,956	2.9	387	193	1.2	1,724	1,054	1.8	8,023	4,203	5.9
Puerto Rico	2,556	13,621	6,307	57.0	2,336	1,098	12.8	7,077	3,262	28.3	23,034	10,667	98.1
TOTAL	317,565	671,098	331,730	14,641.6	321,576	161,404	6,677.2	1,108,026	554,285	13,108.6	2,100,700	1,047,419	34,427.4

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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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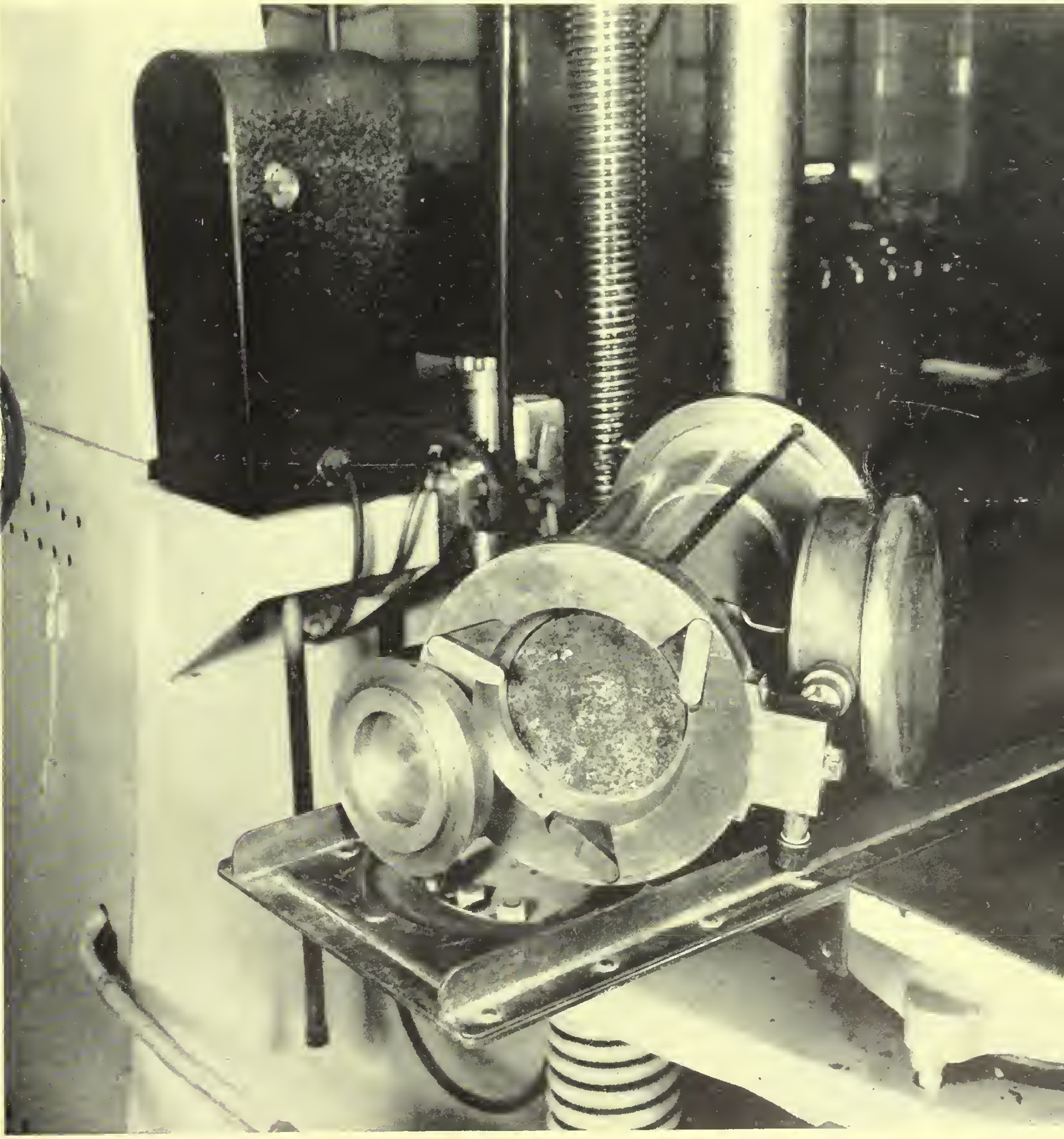

An improved triaxial compression cell, in which specimens are quickly placed and removed through a bottom opening

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BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE
E. A. STROMBERG, Editor

An Improved Triaxial Compression Cell for Testing Bituminous Paving Mixtures

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported[1] **by C. A. CARPENTER, Senior Materials Engineer**
J. F. GOODE, Highway Research Engineer
and R. A. PECK, Assistant Materials Engineer

The triaxial compression test is unquestionably a useful tool in research investigations of bituminous mixtures, but present-day types of cells have two difficulties that hamper their use. The fixed-sleeve triaxial compression cell, with its rubber sleeve attached to the top cell plate, introduces an indeterminate stress component into test results because the top of the sleeve has to stretch as the specimen is deformed vertically. The free-sleeve cell, in which the sleeve is attached to the upper bearing plate, eliminates this indeterminate but must be dismantled for insertion and removal of each specimen.

The Bureau of Public Roads laboratory has now devised a new cell with the advantage of the free-sleeve type in properly measuring stresses, and none of its disadvantages in manipulation. The new cell has a circular opening in the base plate through which the specimen is easily inserted or withdrawn. The plug for the opening, which serves as the bottom bearing plate for the specimen, is quickly attached or released by three latches.

Compressed air is ordinarily used in applying lateral pressure to the specimen, but the new cell is also fitted for high temperature tests with hot water as the heat and pressure-transmitting agent. An added feature is a perforated copper shell to protect the operator if the lucite cylinder bursts under pressure.

THE triaxial compression test for bituminous paving mixtures, through the work of Messrs. Hveem, Endersby, Smith, and others, has been developed to the point where its value as a research tool cannot be doubted. There is little question that it offers a direct and reasonably simple solution of the problem of how to evaluate the two factors, cohesion and internal friction, in bituminous mixtures.

The application of the information obtainable by means of the triaxial test to the problems of investigating the stability of existing pavements and designing new ones of adequate stability is not simple. This has been amply demonstrated by Dr. McLeod in two outstanding papers.[2] [3]

Recognition of the fact that the application of triaxial test data is not a simple process does not in any way constitute a criticism of the tool. On the contrary, it points to a wealth of possible lines of investigation and special application. At the same time, in recognizing the value of the triaxial compression test as a research tool, there is intended no implication prejudicial to the simple, direct tests that have been found useful in the more or less routine work of designing bituminous mixtures in the laboratory.

Present-Day Cells

There are two details, one technical and the other pertaining to convenience of operation, of several present-day triaxial cells that are subject to criticism. It is the purpose of this paper to report on the efforts of the past several years to overcome the difficulties that have given rise to these criticisms.

The first, or technical, criticism arises from the fact that several triaxial cells, in order to provide for maximum convenience of operation, have been designed with full specimen-sized openings through both the top and bottom plates with the rubber sleeve attached to the upper and lower plates around the peripheries of these openings. This type of cell has been called the "fixed-sleeve" type.

From the standpoint of convenience in inserting and removing test specimens, this arrangement is ideal. Unfortunately, however, it introduces in the test results an indeterminate stress component. As the test specimen is deformed by the downward movement of the plunger, the sleeve, being held tightly against it by the surrounding fluid pressure, is prevented from slipping by friction. Thus, the part of the sleeve immediately below the line of attachment to the upper end plate has to stretch just as much as the test specimen is deformed vertically in the test. This stretching cannot be produced without external force. The stronger and tougher the rubber sleeve, the greater is the force required to stretch it as the testing plunger moves down with the deforming specimen. Some of the sleeves now in use, particularly those of neoprene, are extremely strong and resistant to this stretching. Furthermore, there appears to be no satisfactory way of calibrating the equipment in order to correct for the extra force involved, because the length of the stretched section of the sleeve is indefinite and variable.

Several years ago the soils laboratory of the Bureau of Public Roads made an extensive study of the various types of triaxial cells then in use[4] and then designed a very much simplified cell that embodied in one unit what appeared, from a technical standpoint, to be the most desirable features of the several types then in use.

---

[1] Presented at the annual meeting of the Association of Asphalt Paving Technologists, Denver, Colo., Feb. 6, 1951.

[2] *Airport evaluation in Canada*, by Norman W. McLeod. Highway Research Board, Research Report No. 4 B, October 1947.

[3] *The stability of granular and cohesive materials in triaxial compression*, by Norman W. McLeod. Proceedings of the Association of Asphalt Paving Technologists, 1948.

[4] *Essential features of triaxial shear tests*, by C. A. Hogentogler and E. S. Barber. PUBLIC ROADS, vol. 20, No. 7, September 1939.

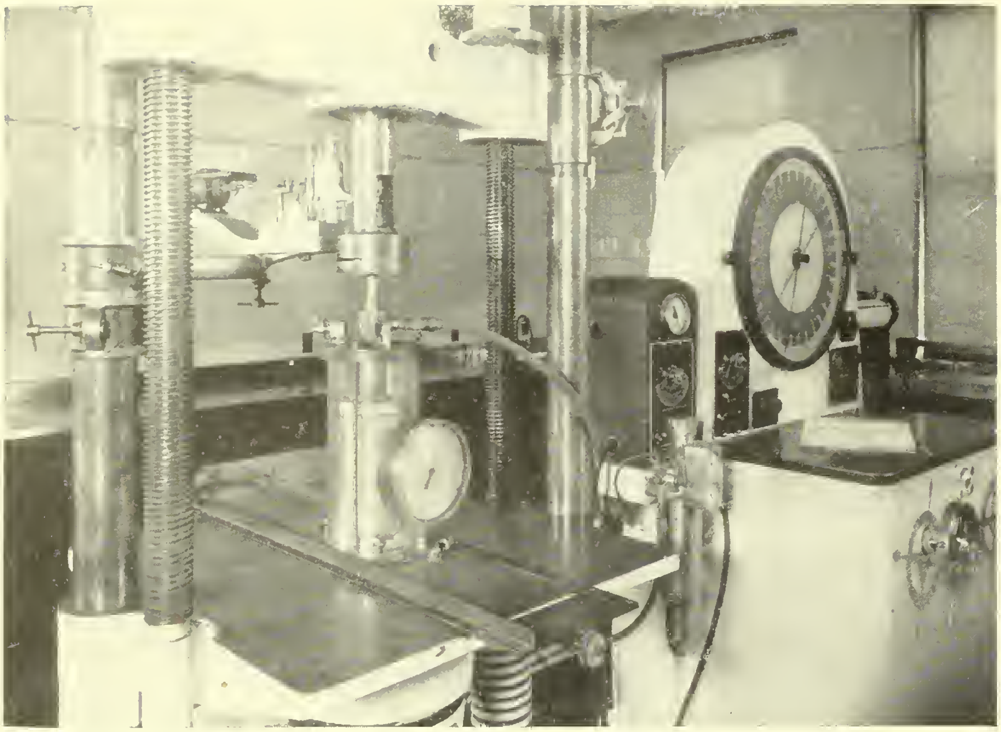

Figure 1.—The cell, with specimen in place, ready for test. Note the compressometer above.

Most important of the features incorporated in the Public Roads design was the so-called free rubber sleeve. In the free-sleeve type of cell, the upper end of the rubber sleeve is attached not to the top plate of the cell but to an upper bearing plate, which was designed to operate entirely within the pressurized zone. This eliminated the longitudinal stretching of the sleeve and the indeterminate vertical force component that the stretching entailed.

Much of the simplification of design that was attained was made possible by adopting air, rather than liquid, for the cell pressure medium. The air is taken from a compressor and passed through a flow-type back-pressure regulator which delivers it at the desired gage pressure. The desired pressure is easily maintained by simple manipulation of the pressure-regulator controls even when there is considerable leakage of air from the cell.

The second criticism, applicable to the free-sleeve cells, is based on operational difficulties. The criticism arose from the inherent characteristics of these cells: namely, that it was necessary to disassemble the cell and disconnect the sleeve from the interior, upper bearing plate in order to insert or remove each test specimen. The Public Roads cell was designed so that this involved a minimum of inconvenience, but, when utilized for testing bituminous mixtures, it was found that a modification of design would be advantageous for this special application in order to facilitate the testing of a larger number of specimens in a given time. The desired result has been obtained by designing a new cell which provides for the insertion and removal of test specimens through an opening in the base plate.

The New Public Roads Cell

Figure 2 is a diagrammatic sketch of the new cell. As in all free-sleeve types, the rubber sleeve is attached to the testing plunger. In this model, the testing plunger or plate is positioned within the cell and actuated by a polished stainless-steel thrust rod which operates through a lap-finished bronze bushing in the top plate of the cell.

The new and special feature of this cell is the opening in the base plate, which permits the insertion of test specimens directly into the sleeve, and also their removal, without disassembling the cell. Thus all the operational advantages of the fixed-sleeve type cell are gained, while still retaining the technically desirable free sleeve which permits the testing of the specimens without developing the indeterminate stress component mentioned above.

The bottom of the rubber sleeve is attached to the lower plate of the cell around the periphery of the opening by means of two interlocking clamping rings in a manner not essentially different from that used in some of the fixed-sleeve cells. The base

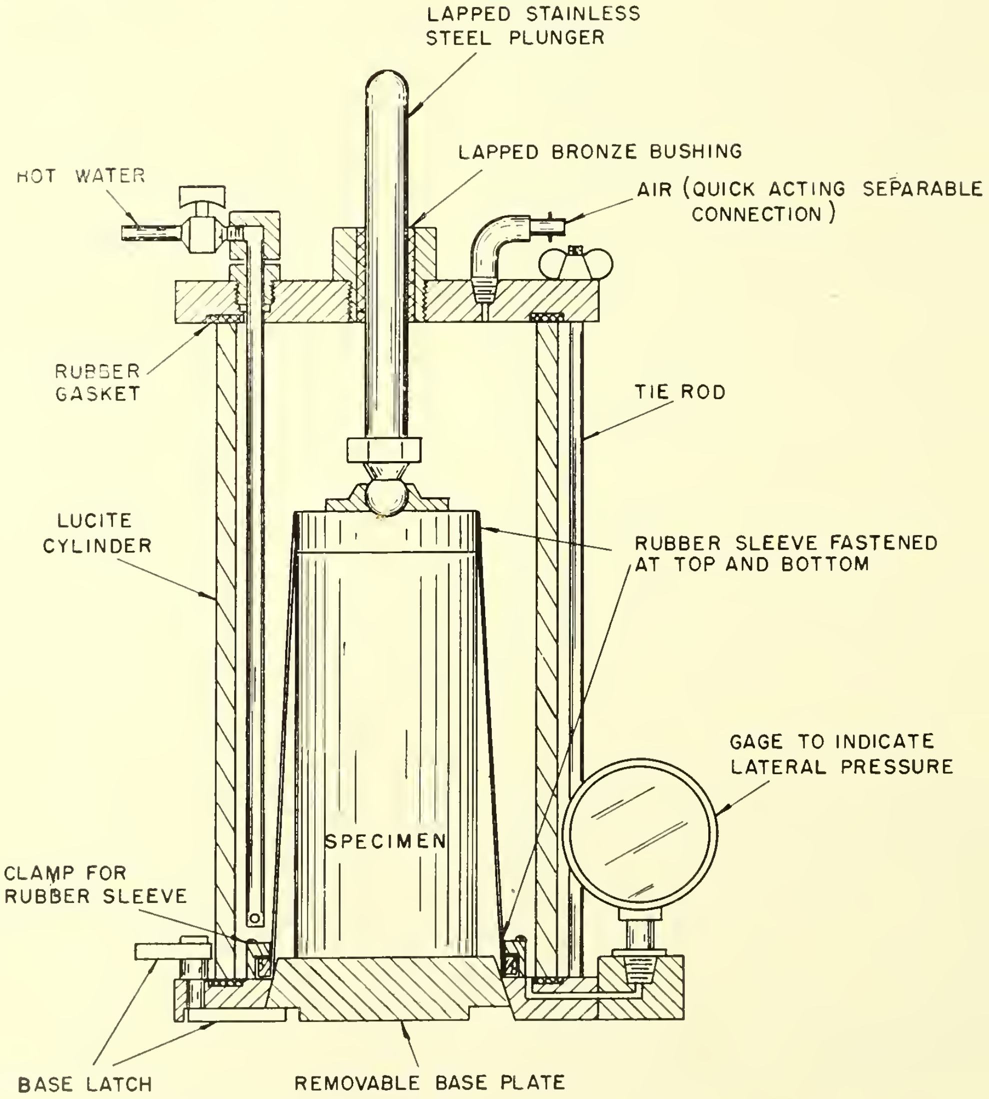
LAPPED STAINLESS STEEL PLUNGER
LAPPED BRONZE BUSHING
HOT WATER
AIR (QUICK ACTING SEPARABLE CONNECTION)
RUBBER GASKET
TIE ROD
LUCITE CYLINDER
RUBBER SLEEVE FASTENED AT TOP AND BOTTOM
GAGE TO INDICATE LATERAL PRESSURE
SPECIMEN
CLAMP FOR RUBBER SLEEVE
BASE LATCH
REMOVABLE BASE PLATE

Figure 2.—Diagrammatic sketch of improved triaxial cell.

plug or rest for the specimen is not provided with a sealing gasket and, therefore, does not provide full maintenance of pore pressure when testing water-saturated specimens. In general, the maintenance of pore pressure in testing saturated bituminous mixtures is not desired. If needed, the design can be modified to provide for sealing the plug so that pore pressure will be maintained.

A characteristic of all free-sleeve cells that probably should be mentioned here is that the cell pressure is applied not only laterally on the test specimen, but also over the top except for the relatively small area of the thrust rod (see fig. 2). Because of this fact it is necessary, before computing the values for cohesion and friction, to correct the indicated total vertical load by adding to it the quantity, unit cell pressure times net area of bearing plate.

Air or Water Pressure Used

As long as the test is made only at laboratory room temperature with the specimen brought to 77° F. before inserting it in the cell, air is the most convenient fluid for use in applying the lateral pressure. Air does not cause rusting of the equipment, its leakage leaves no liquid to dispose of, and it is usually more conveniently available under suitably controlled pressure. However, in order to provide for at least some investigational testing at higher temperatures, provision has been made for introducing hot water into the cell and for determining the temperature of this water at the time the compression test is run.

In the present design, the hot water is supplied directly from a hot-cold mixing tap through a hose to the hot water fitting at the upper left (see fig. 2). The cell is filled with water at a temperature just slightly above the desired test temperature. The water is allowed to cool to test temperature as indicated by a thermometer fastened to the inner surface of the lucite cylinder. The specimen is then inserted, the desired pressure is applied by means of air acting on top of the water, and the compression test is made. Regardless of what predetermined test temperature is being used, all test specimens are brought to that temperature in an automatically controlled air bath or oven before being inserted in the cell. This method of testing at elevated temperatures takes somewhat longer than would be necessary if it were possible to circulate water at the correct temperature continuously through the cell because, in the present set-up, the water has to be blown out and replaced with warmer water as often as the temperature falls noticeably below the desired test temperature. This has not proved to be particularly inconvenient, however, because it takes so little time to insert a specimen and make a triaxial test in this equipment. Hence, it is possible to complete several tests before the water has to be changed.

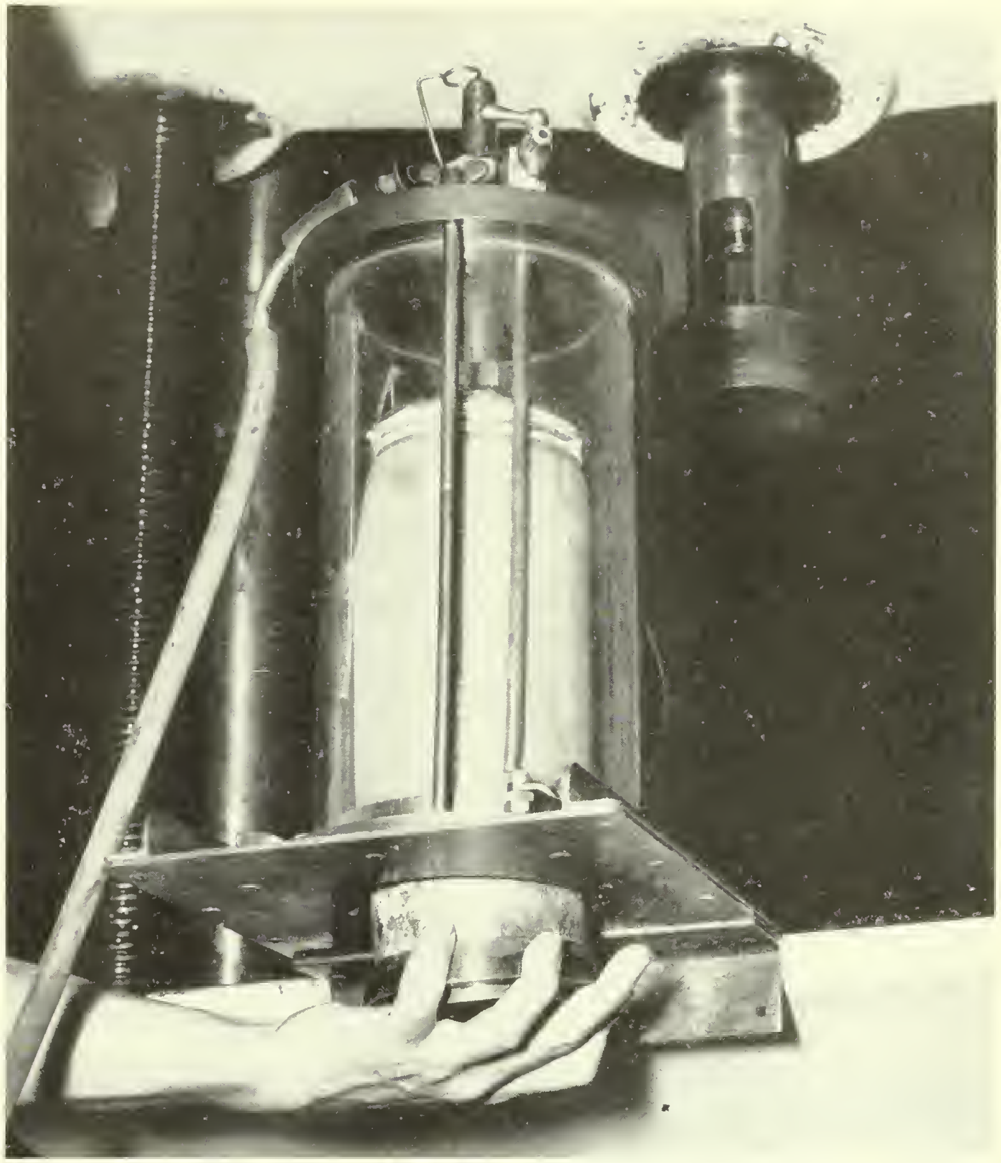

Figure 3.—Inserting a specimen into the improved cell, which rests on the detachable shelf.

The Cell in Use

The cover photograph shows the cell lying on its side, with a specimen in place for testing. The bottom plug is at the left, leaning against the base plate of the cell. The three latching bars are necessary to prevent the air pressure from blowing the specimen and plug back out through the bottom opening and thus lifting the entire cell off the platen of the testing machine.

The cell is shown resting on a special bracket and centering guide that may be attached to or removed from the testing machine in a matter of seconds. The bracket shelf has an opening through it large enough for the test specimen and the plug to pass through freely and is so arranged that the cell can be slid easily directly onto it from the platen of the testing machine or from the shelf onto the machine in a vertical position. This arrangement adds materially to the convenience and speed of operation.

Figure 3 shows the operation of inserting a test specimen or of removing one. Note that the rubber sleeve is considerably enlarged to permit free movement of the specimen. This enlargement of the sleeve is obtained by drawing a slight vacuum on the cell through the rubber tube and needle valve at the left and top, respectively. In this view, one of the bottom latch levers is shown just above the edge of the bracket shelf and slightly to the right of center of the cell. The vertical steel rod in the foreground is one of the three tie rods with top wing nuts which provide for quick disassembly of the cell for cleaning and drying. Just to the right of the tie rod and inside the cell, between the lucite cylinder wall and the rubber sleeve, is a copper tube with one bottom side opening and an external stop cock at the top. This assembly is provided for introducing and removing hot water when tests are to be made at elevated temperatures. It does not have any function when air alone is used as the cell-pressure medium.

Replacement Sleeves

Replacement sleeves are made up in the laboratory as often as needed. The stock material is dental dam rubber which is purchased in sheets 0.01 inch thick, 36 inches wide, and in any length desired. To make a sleeve for a 4- by 8-inch test specimen, a strip of rubber about 10½ inches wide and 36 inches long is cut from the stock sheet, attached along all edges to a supporting frame, and coated on portions of both sides with one coat of cold patching rubber cement. The supporting frame has an opening 10½ inches wide and 38 inches long. The rubber is attached to the frame by means of adhesive tape, some stretching of the sheet being necessary in order to accomplish this. To simplify the following discussion, one end of the rubber sheet will be referred to as end *A* and the other *B*. In applying the rubber cement, the upper side is coated from end *A*, a distance of about 25½ inches toward end *B*, leaving the last 12½ inches toward end *B* uncoated. The under side is coated from end *B* toward end *A*, a distance of 25½ inches. After the cement has dried thoroughly in the air, the adhesive tape holding end *B* to the frame is removed. A 4-inch-diameter mandrel is laid on this end, which is uncoated on its upper side. The rubber sheet is then fastened with adhesive tape to the face of the mandrel. Then the mandrel is rolled toward the other end, pulling the rubber loose from the adhesive tape on the edges of the frame as it goes along. Three complete wraps are made and the material provides sufficient overlap to produce a four-ply joint. The result is a three-ply rubber sleeve which, because of its laminated structure, is extremely tough and durable. Because of its thinness and the fact that it is composed of new, live natural rubber, the sleeve is so resilient that it has no appreciable stiffness and has, therefore, no detectable effect on the test results that can be attributed to its physical character.

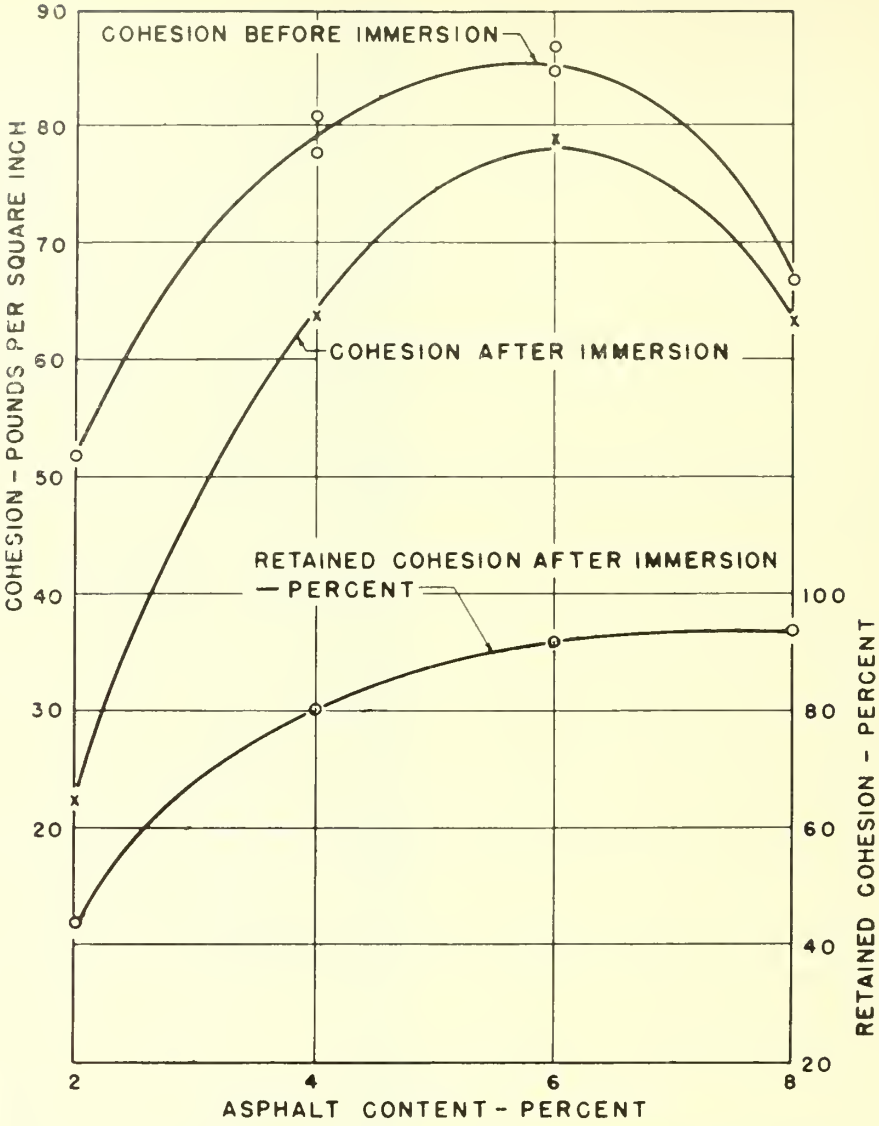

Figure 5.—Effect of asphalt content on cohesion and secondary effect of immersion.

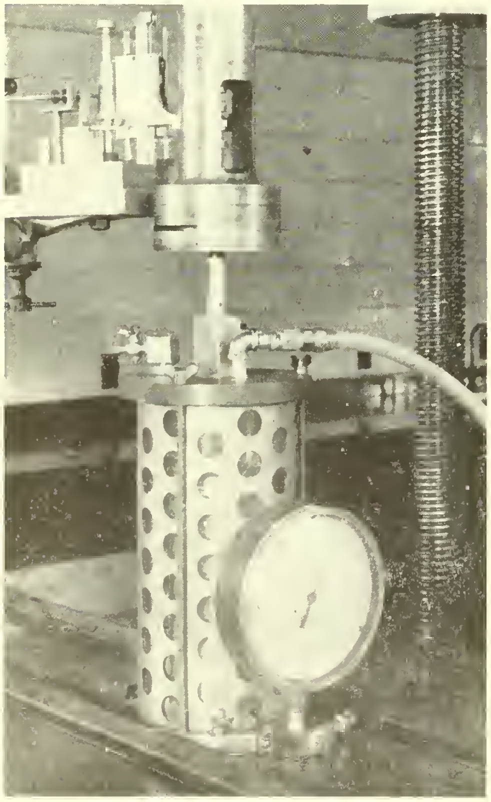

Figure 4.—The triaxial cell encased in its protective perforated copper guard.

Because, in wrapping the cement-coated sheet on the mandrel, one of the uncoated ends is applied first and the other uncoated end goes on last, the inside and the outside of the finished sleeve are not fouled with unneeded cement. The operation of removing the sheet from the supporting frame and wrapping it on the mandrel requires considerable care to prevent folding, wrinkling, or creasing, and to maintain a suitable tension while the rolling is being done. The first few attempts are quite apt to be disappointing, but it will be found that a satisfactory technique can be developed and that the resulting product is highly satisfactory from a technical standpoint.

To remove the finished sleeve from the mandrel it is simply rolled back at one end and turned wrong side out, the strip of tape being pulled loose from the mandrel in the process. Final preparation of the sleeve for installation in the cell consists of trimming off the inside flap formed by the tape and squaring the ends, also by trimming.

The sleeve is attached to the upper plate or plunger of the cell by first cementing and then wrapping with cotton cord or large rubber bands. As previously noted, it is attached around the bottom plate opening with two interlocking clamping rings which are screwed to the bed plate, the folds of the sleeve around the lower clamping ring constituting the sealing gasket. The extra 2½ inches of sleeve length provided by cutting the original

Table 1.—*Effect of varying the asphalt content of bituminous mixtures on cohesion, friction, and unconfined compressive strength before and after immersion*

Parts asphalt per 100 of aggregate	Results [1] of tests—						Retention of characteristics after immersion	
	On dry specimens			On wet specimens [2]				
	Unconfined compression	Cohesion	Friction	Unconfined compression	Cohesion	Friction	Retained strength [3]	Retained cohesion [4]
	P.s.i.	*P.s.i.*	*Tan ø*	*P.s.i.*	*P.s.i.*	*Tan ø*	*Percent*	*Percent*
2	290	52	1.19					
2	268			113	23	1.08	41	44
4	374	81	.96					
4	384	78	1.03					
4	379			248	64	.74	65	81
6	376	87	.83					
6	369	85	.85					
6	392			276	79	.61	73	92
8	258	67	.70					
8	248			220	64	.56	87	96

[1] Averages of three determinations.
[2] After immersion in water at 120° F. for 4 days.
[3] Retained strength =(strength after immersion ÷ average of dry strengths for same mixture) × 100.
[4] Retained cohesion = (cohesion after immersion ÷ average of dry cohesions for same mixture) × 100.

strip 10½ inches instead of 8 inches wide are needed for making these connections and for trimming.

Testing Practice

Figure 1 shows the cell in place on the platen of the testing machine with the air-pressure hose from the back-pressure regulator connected at the upper right of the cell and the pressure gage mounted on the base of the cell. Tests are currently being made at cell pressures of 0, 30, and 60 pounds per square inch. Also shown in figure 1 is the compressometer that is used in obtaining load-deformation recordings of all the tests. The feeler of the compressometer is actuated by a polished bearing face in the stem of the upper bearing plate of the testing machine on a line which is an extension of the vertical axis of the test specimen. The recording drum may be seen at the right of the large load indicator dial.

In conducting triaxial tests in the Public Roads laboratory, as well as all other compressive tests of bituminous paving mixtures, the specimens are stressed to failure by continuously increasing loading and, therefore, continuously increasing deformation. A rate of vertical deformation of 0.05 inch per minute per inch of specimen height is used. Thus, if the test specimen is 8 inches high, the platen speed is 0.4 inch per minute and for a 4-inch specimen it is 0.2 inch per minute. Although this unit rate of deformation was selected somewhat arbitrarily and possibly has no superiority over any other reasonable rate, it is extremely important that all test results that are to be compared with one another for evaluation of one or more characteristics should be obtained under some inflexible set of standard conditions. The plastic or semi-plastic bituminous mixtures are especially sensitive to such factors as rate of deformation and temperature. High rates of deformation result in high cohesion values and high compressive strength values, while high temperatures have the opposite effect and result in low cohesion and compressive strength values. Therefore, the greatest care must be exercised to maintain the rate of deformation and the specimen temperature as nearly constant as possible during the tests.

Up to this point, the features and techniques having to do primarily with the production of satisfactory test data have been considered. Before presenting illustrative test data, mention should be made of the difficulty that was experienced in the early use of the triaxial test for bituminous mixtures because of the adhesion of the specimen to the rubber sleeve and to the upper bearing plate or plunger. A simple solution for this trouble was found to be a light dusting of the specimen with powdered talc just before making the test. This practice has not only eliminated the operational difficulties, but has added materially to the useful life of the rubber sleeve, first, by keeping it free of bituminous material which tended to soften it eventually and second, by protecting it to some extent from being punctured or ruptured by specimens having abrasive or porous surfaces.

A Cell Guard

A recent addition to the cell has been made to guard the operator against injury in case of failure of the lucite cylinder to withstand the internal air pressure. Recently, an operator in a State laboratory

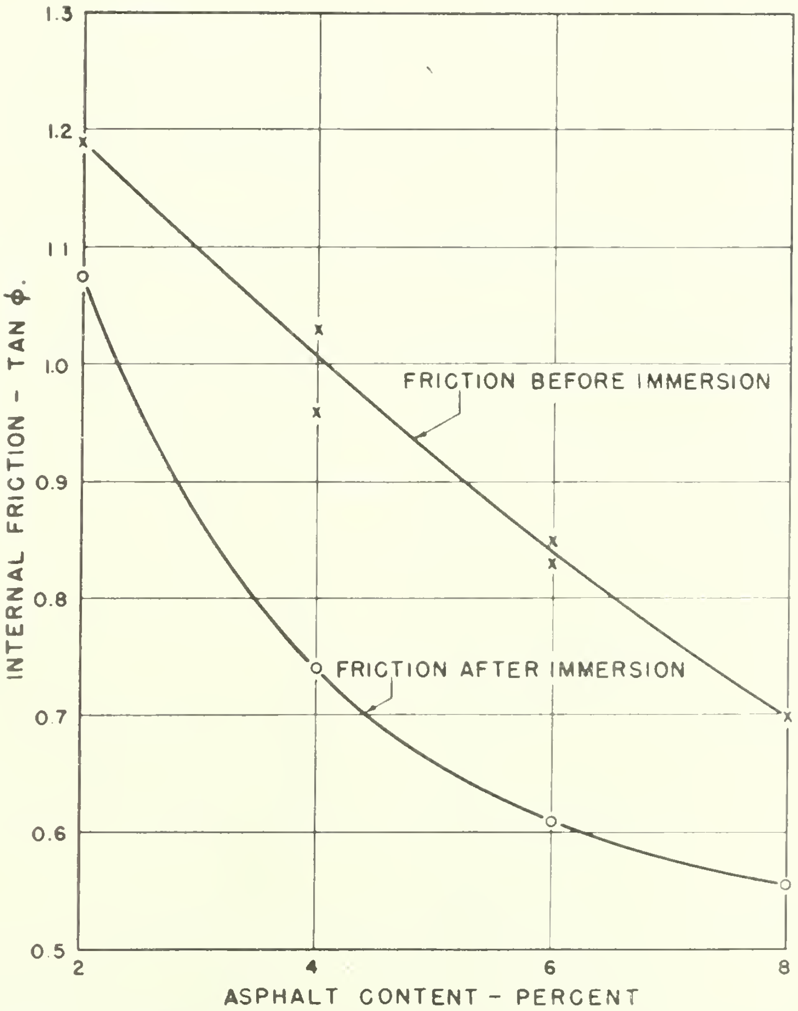

Figure 6.—Effect of asphalt content on internal friction and secondary effect of immersion.

was seriously injured by flying particles of lucite when the internal air pressure exploded the cylinder. The use of compressed air always entails the danger of an explosive rupture of the equipment. This danger is not present when a liquid pressure medium is used, unless the liquid is pressurized with compressed air. Figure 4 shows the protective addition—a perforated sheet-copper shield that has been placed around the lucite cylinder to cushion a possible rupture and to retard the flight of broken pieces of lucite. It is believed that this is an important precaution that should be taken even though the equipment may have been pretested at a pressure considerably higher than that to be used in normal operation.

Of course, the lucite might be replaced by a brass or steel cylinder, but the transparent cylinder has the definite advantage of permitting observation of test conditions, some of which have a bearing on the results, and the perforated copper shield does not interfere seriously with this feature of the lucite cylinder.

Preliminary Tests With New Cell

While the primary purpose of this paper is to describe the equipment that has been developed in the Public Roads laboratory for the triaxial compression testing of bituminous paving mixtures, it will be of interest to present some data that have been developed in order to convey an idea of how successfully the present equipment performs.

In table 1, there are shown the data resulting from a preliminary investigation of the effect of variations in asphalt content on the cohesion and friction values and on the unconfined compression values for a series of bituminous mixtures that were identical in all respects except asphalt content. A secondary purpose of this study was to determine the effect, on the triaxial and unconfined compression test values, of immersing the same series of mixtures in water at 120° F. for 4 days.

The aggregate used in all the mixtures was a combination of crushed granite having little resistance to the stripping action of water, Potomac River sand, and limestone dust. The various aggregate fractions were weighed out in batches, each of just sufficient quantity to form one 4- by 4-inch test cylinder. These individual aggregate batches were preheated to the mixing temperature of approximately 325° F. in an automatically controlled electric oven and then mixed with the asphalt in a special laboratory mixer having a maximum capacity of slightly more than 2,000 grams. The asphalt was an 85–100 penetration grade having an actual penetration of 95. It was brought to the mixing temperature just before adding it to the aggregate in the mixer and only enough asphalt was heated at one time to make one batch. The mixtures were compacted in a double-plunger cylindrical mold under a load of 3,000 pounds per square inch after first being carefully spaded around the edge and leveled in the mold to reduce surface voids and local strains during compression. This initial spading and leveling with a thin-bladed spatula has been employed for several years and has been found to aid materially not only in producing specimens of more uniform appearance, but specimens having more uniform densities and less aggregate degradation than was the case before it was adopted. The 4- by 4-inch specimens were used because it was desired to include the previously mentioned study of the effect of water and it had been found that 4- by 8-inch specimens were very difficult to handle after immersion without deforming them and rendering the test results somewhat erratic. All compression tests were made at 77°F. on both dry and wet groups.

Test Results

The data shown in table 1 have been plotted in figures 5–8 to show graphically the various relations to be investigated. Figure 5 shows the effect on the cohesion values resulting from varying the asphalt content from 2 to 8 parts by weight per 100 parts of aggregate, both before and after immersion in water. In the bottom curve, the gradually increasing tendency of the richer mixtures to resist the action of the water is shown by plotting the percentage of the cohesion of the dry specimens that was retained after the immersion period, against the asphalt content of the mixtures. The rise of the cohesion curves for both the dry and wet specimens from relatively low values for the lean mixtures to optimum values for some intermediate asphalt content and then the quick falling off of the cohesion values for the excessively rich mixture are characteristic. Of special interest is the indication, which might be expected, that the lean mixtures may be very susceptible to the action of water, while the 8-percent mixture, having its pore spaces virtually sealed or filled with asphalt, is almost unaffected by immersion insofar as the cohesion reflects this effect.

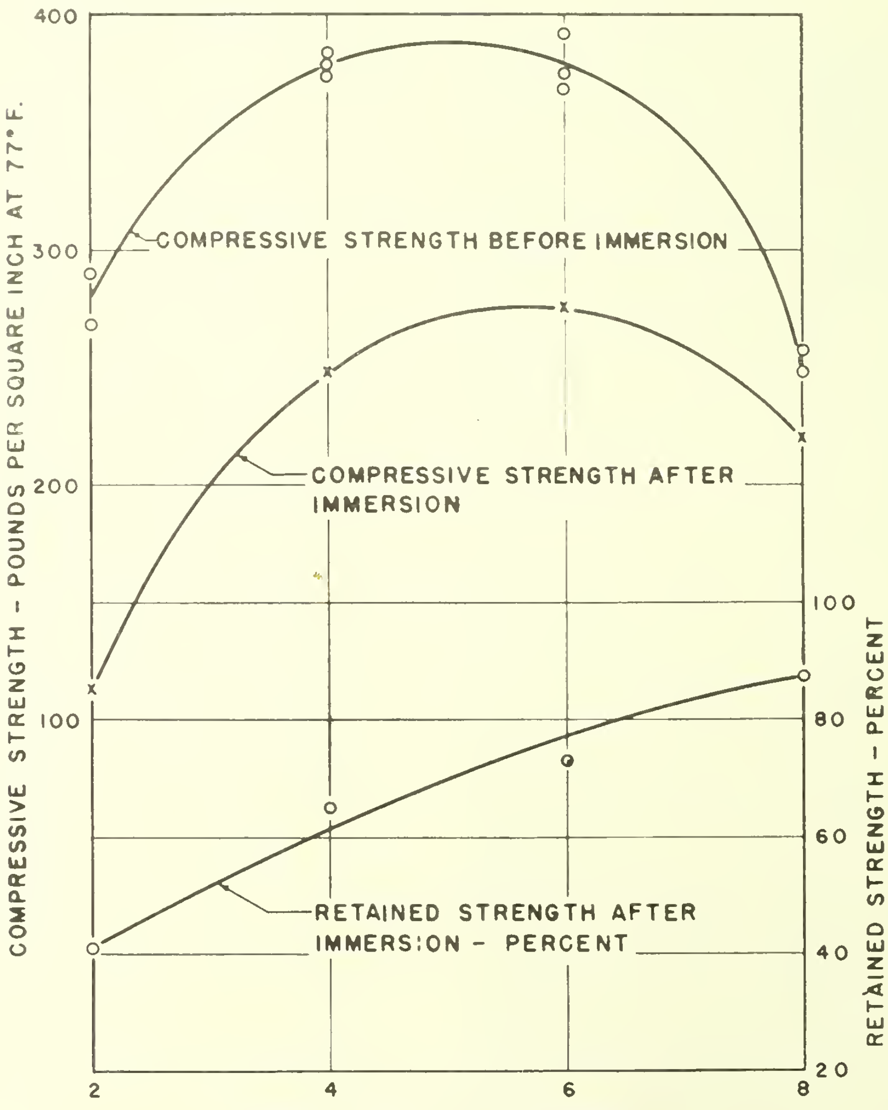

Figure 7.—Effect of asphalt content on unconfined compressive strength and secondary effect of immersion.

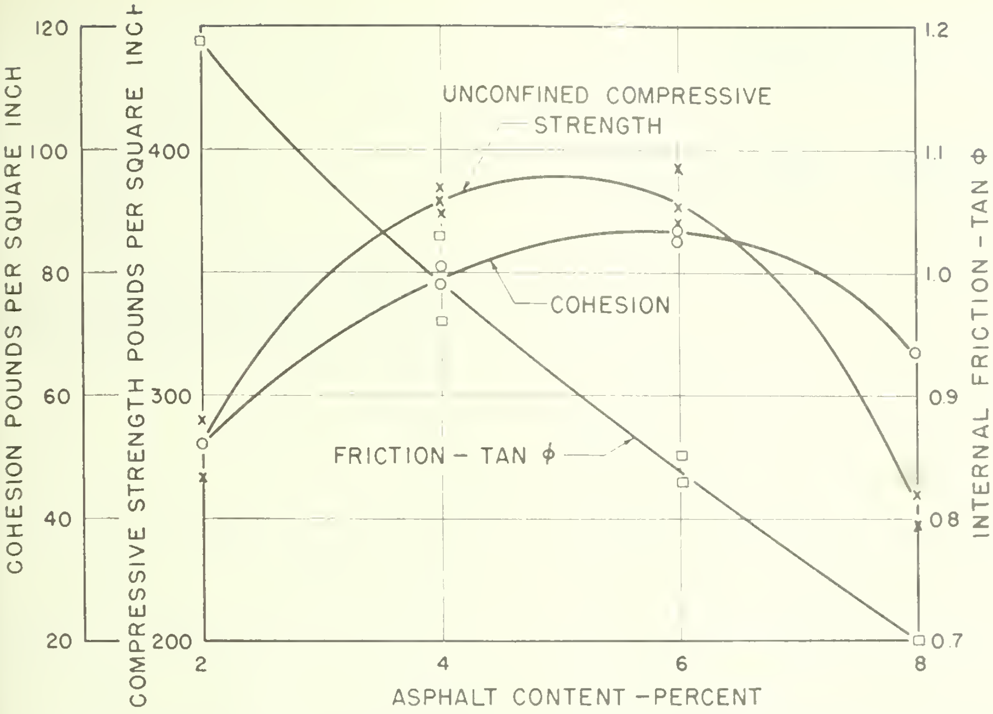

Figure 8.—Effect of asphalt content on cohesion, unconfined compressive strength, and internal friction.

Figure 6 shows the effect of increasing asphalt content in reducing, at an almost uniform rate, the internal friction of the mixtures prior to immersion. On superficial consideration of the lower curve, showing the additional loss in friction when the specimens were immersed, it might seem illogical that the very lean mixture is affected much less than any of the others, including those of intermediate or near-optimum asphalt content.

On more careful analysis, it appears that this might be explained in this way: The lean mixture contains so little asphalt that its internal friction is almost the same as that of the aggregate itself. However, it makes little difference how much water enters the mixture, insofar as the effect on friction is concerned, because water has such a low viscosity that without pore pressure it has almost no effect on the friction of granular aggregates. The mixtures of intermediate asphalt content still have enough pore spaces to admit considerable amounts of water, but the pore openings are probably enough restricted by asphalt so that, when the specimens are loaded, the water creates some pore pressure as shearing occurs and thus affects the internal friction to a marked degree. The very rich mixture admits little water and, therefore, as compared to the mixtures before immersion, is less affected than the intermediate ones. It seems fairly safe to assume that a little more asphalt would completely seal out the water and, at some point to the right, the two curves might become one. At such a point both the cohesion and internal friction would be so low that the mixture would have only as much supporting value as would be possessed by asphalt alone.

Figure 7 shows relations between unconfined compressive strength and asphalt content that are strikingly similar to the cohesion curves of figure 5, probably indicating that the unconfined compression values are affected more by the cohesion factor than by the internal friction. This is also indicated in figure 8 where the cohesion, unconfined compression, and friction values for the water-free specimens are shown in their relation to each other. However, the influence of friction is also to be seen in the compression values at the right, in that as soon as the cohesion values pass the maximum and begin to fall off the compression curve falls at an increasing rate, as if drawn downward by the rapidly falling friction curve.

While the test data that have been presented are not extensive, it being the purpose of this paper to describe certain developments in the equipment and techniques rather than to report extensively on the application, it is felt that they are consistent and indicative of the characteristics of the materials. In most respects, the data tend to corroborate the findings of other investigators who have used essentially similar equipment.

As to the choice of cell type, it appears that any one of several that are in use today can be used successfully. The widespread interest in the triaxial test and the numerous very stimulating papers and reports that have been written recently on the subject indicate that the triaxial test is definitely a valuable research tool. Its demonstrated value as such will justify considerable effort to select or develop the most convenient and dependable model possible and then to standardize it so that those who use the triaxial method of test may speak the same language when they want to discuss the factors that affect cohesion and internal friction.

Highway Control Sections
Their Establishment and Use

BY THE FINANCIAL AND
ADMINISTRATIVE RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by GORDON D. GRONBERG,
Highway Economist

The division of a highway system into permanently established highway control sections provides logical, practical, and convenient units for which a vast amount of highway cost and performance data may be recorded in an organized fashion. The use of a uniform accounting system is important if the data so compiled are to be of value.

Cost data obtained from control sections are used in the preparation of highway construction and maintenance budgets and for analyses and comparisons of construction and maintenance costs by surface types, roadway widths, surface thicknesses, year of construction, and topographic locations. Special investigations have been undertaken by control sections to study soil conditions, influence of traffic on various surface types, costs per vehicle-mile, and construction and maintenance economics.

THERE exists today in every State a great need for a centrally located and organized body of data relating to every phase of highway management, operation, and performance. The increasing demands for all kinds of information for planning, programing, administrative, and legislative purposes have highlighted this need. The most effective means of accomplishing the assembly of such a vast amount of data in an organized fashion is through the establishment of highway control sections.[1]

In essence, the control section procedure involves the division of the highway system into convenient, practical units for which design, construction and maintenance cost, inventory, and performance records are kept on a uniform and continuing basis. Once established, the units are permanently fixed so that records maintained through the years will be directly comparable. The control section system described in this article has already been adopted by 17 States.

One of the problems that has confronted the highway administrator has been the recording of highway expenditures in a manner that will permit comparisons of construction and maintenance costs for various sections of the highway system. It has generally been impossible, in the past, to correlate construction costs with maintenance costs, as maintenance sections were often consolidated, eliminated, or adjusted from year to year. The establishment of control sections and the reporting of construction and maintenance costs on a uniform basis will make these comparisons possible.

Uniform construction and maintenance operation and cost records kept for permanently designated control sections enable States to make comparisons of costs based on similar pieces of property; factual information can be obtained on various kinds of highway work as affected by climate, traffic, etc.; and studies can be undertaken on the annual costs for each control section.[2] The information is also of fundamental value in the management of a highway department on an efficient basis.

Nature of Control Sections

Highway control sections are segments of highway whose termini are county lines and major intersecting routes. In certain cases, other criteria such as changes in topography (mountainous or non-mountainous), traffic characteristics, and adaptability as maintenance sections have been used. Where a bridge is encountered at the termini of two control sections or at a county line, the bridge and approaches are usually included in their entirety in one or the other control section. In some States, because of shared financial responsibility, etc., it has been necessary to distribute a proportionate share of the cost of such bridges to each county. Major structures, interchanges, traffic circles, Y connections, ramps, etc., are included either with the control section on the main or through route or with the route on which they were constructed. These special highway facilities can also be set up as separate control sections, if desired.

The essential attributes of a desirable control section are fourfold. The control section should be, first, a unit with a reasonably uniform traffic volume throughout its length; second, a logical unit for development to the same general type and standard; third, a practical unit for reporting maintenance costs; and fourth, a convenient unit for the compilation of statistical and research data involving the assembly and correlation of construction and maintenance expenses, service-life characteristics, traffic data, design standards, etc.

The length of control sections varies, depending on the frequency of intersections, changes in topography, etc. In the eastern part of the United States the average length is 5 to 8 miles, in the central part from 10 to 12 miles, and in the western part from 15 to 18 miles.

Once established, control sections are permanent units of the highway and are not to be changed except as necessitated by relocation or where extensions of or additions to the highway system are encountered in the future. These control sections are utilized by each division of the highway department—planning, programing, right-of-way, design, construction, maintenance, and accounting.

Control sections have already been adopted in 17 States, and are under discussion in many of the remaining States. Control sections are established on the entire State highway system. In a few States they have also been established on the State-aid roads in addition to the State highway system. It is of interest to note that control sections have been put into use in Puerto Rico and in Turkey.

The general objectives of the control section procedure are clear cut and should not be made complicated or rendered burdensome by the incorporation of too much detail. Further, before control sections are put into operation the field personnel should be carefully schooled and fully instructed in the most practical methods of reporting operations and costs.

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[1] *Highway control sections*, by Fred B. Farrell. American Highways, vol. 25, No. 1, January 1946. Published by the American Association of State Highway Officials. *Also*, Report of the Committee on Highway Costs, Department of Economics, Finance, and Administration, Highway Research Board. Highway Research Correlation Service Circular No. 61, May 1949.

[2] *A procedure for determining the annual cost of a section of rural highway*, by H. W. Hansen. PUBLIC ROADS, vol. 26, No. 7, April 1951.

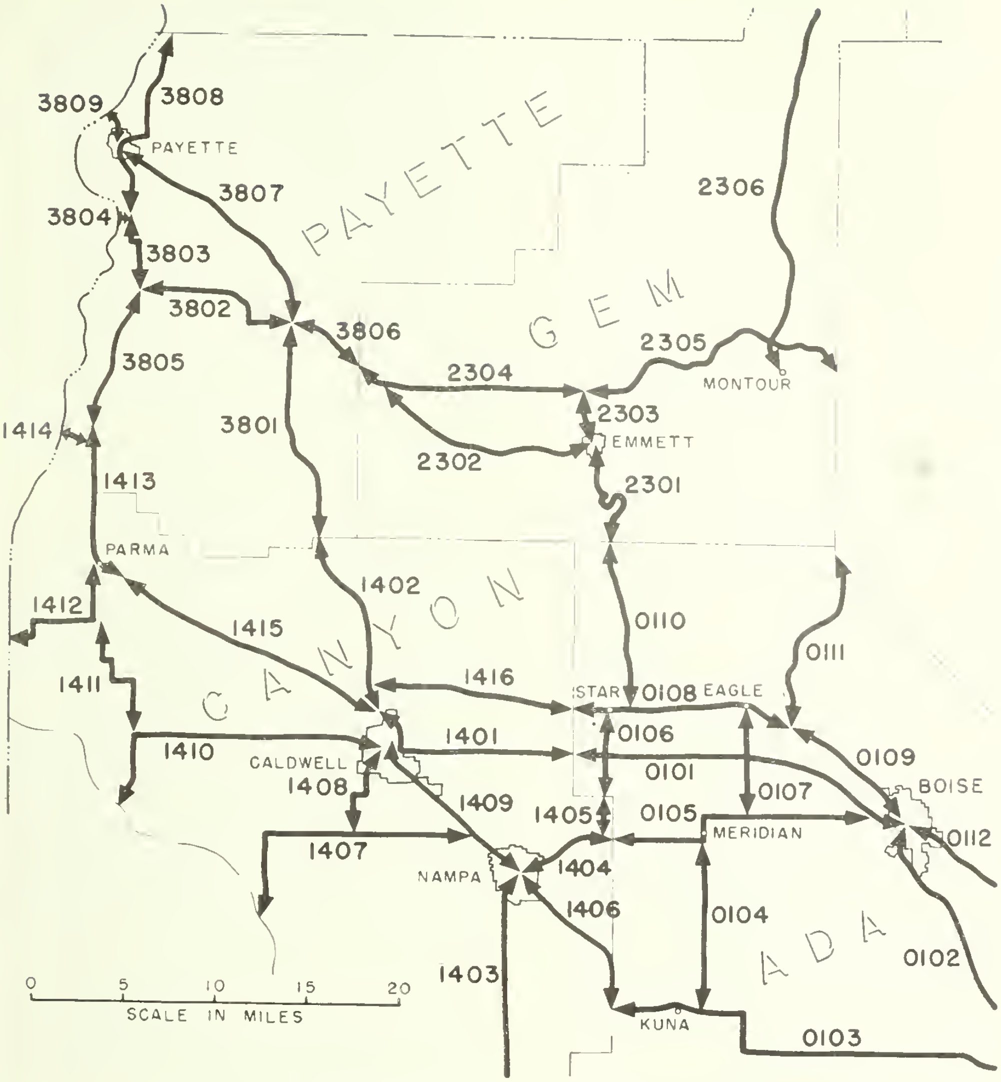

Figure 1.—A portion of the control section system in Idaho.

Establishing Control Sections

For an average State the establishment of control sections on the State highway system, involving selection and designation of termini and other preliminary work necessary in outlining a general plan of operation, can usually be accomplished in 2 or 3 weeks. In making the change-over to the new procedure, the work of drafting control sections on county maps, preparing control section descriptions, revising pay roll and other forms, and instructing field personnel in the use of new forms and coding procedures ordinarily takes from 6 to 8 months.

A control section committee, composed of the heads of the various divisions of the highway department, is usually organized when a State undertakes the establishment of control sections. Any proposed changes in control section termini or the addition of new sections due to relocations or additions to the system are subject to the review and approval of the committee. It is also the function of this committee to supervise the drafting of control sections on county maps, to write control section descriptions, to prepare manuals of instruction for reporting costs by uniform cost classification, and to coordinate its activities with those of the Bureau of Public Roads in Federal-aid route and project numbering and other matters of joint interest.

Once control sections have been established, a special project under the highway planning division of the State highway department can be set up to bring the historical construction data, particularly construction costs, up to current status on a control section basis for use in various types of economic road cost analyses.

Numbering and Termini

It is general practice to use a four- to six-digit number to identify the control section. The first two or three digits usually indicate the county number in alphabetical order, and the last two or three digits indicate the section number within the county.

Figure 1 shows a portion of the control-section set-up established in Idaho. These control sections are a fixed series of road sections upon which all construction costs, maintenance costs, traffic, and other operational data are kept by the State as a matter of permanent record.

Figure 2 illustrates the suggested procedure for handling control section termini at rural road intersections. As shown at the left in the figure, where the main route intersects a secondary route, the main route is continuous and the secondary route breaks at the intersection. As shown at the right in the figure, where the main route intersects a more important secondary route, the volume of traffic at the intersection warrants a break in both routes. This can be handled by dividing the main route into two control sections with a Y connection in each section, or by treating the entire intersection as a separate control section.

The principle of control sections can also be applied to city streets, in which case each block and each intersection becomes a control section.[3] Figure 3 illustrates the method of establishing control sections in cities.

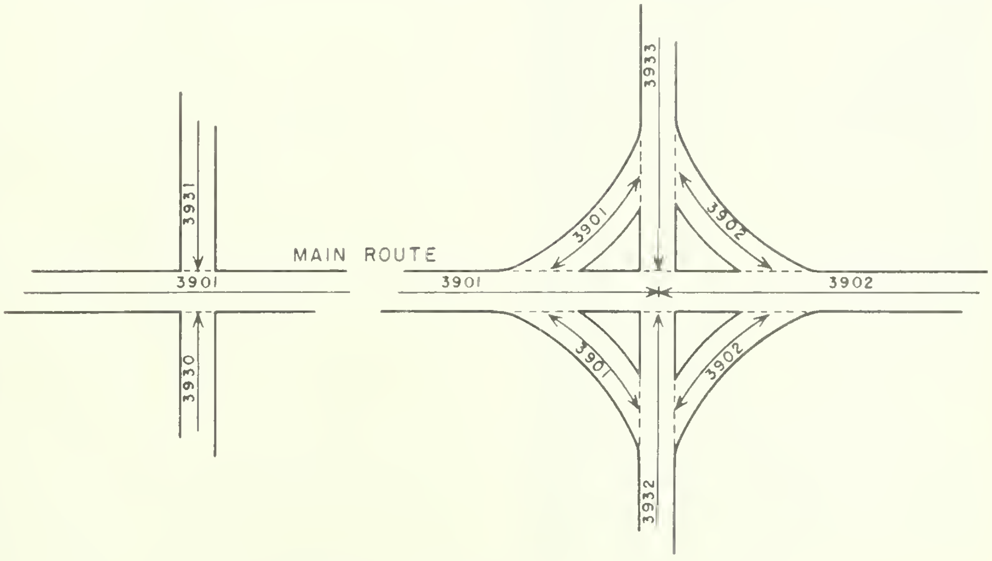

Figure 2.—Control section termini at rural intersections.

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[3] *Milwaukee methods of determining and controlling lengths of blocks and intersections,* by R. W. Gamble. Proceedings of the Highway Research Board, vol. 26, 1946.

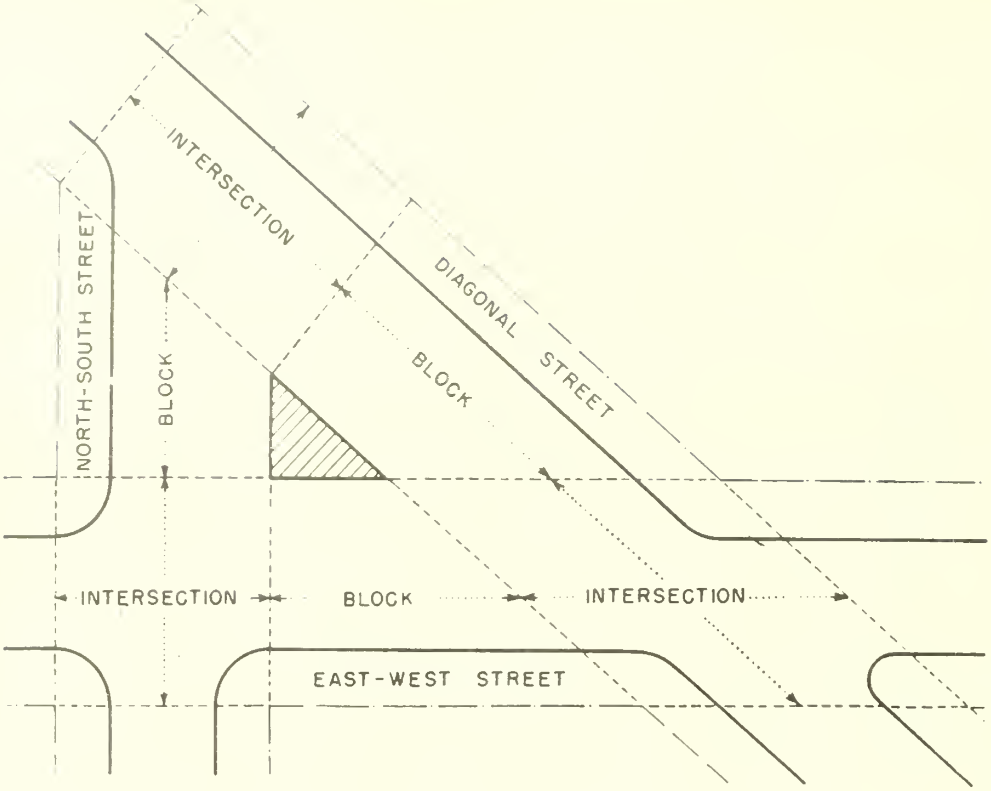

Figure 3.—Control section termini in a city.

Figures 4 and 5 are examples of the methods that have been used by various States to physically designate control section termini. In Maine (fig. 4), the control section number that can be seen from the road is the section being entered. In Nevada (fig. 5), the posts are set at an angle and both control section numbers are visible from the road. Control sections in Nevada are designated by two letters and two numbers, the former identifying the county and the latter identifying the control section within that county.

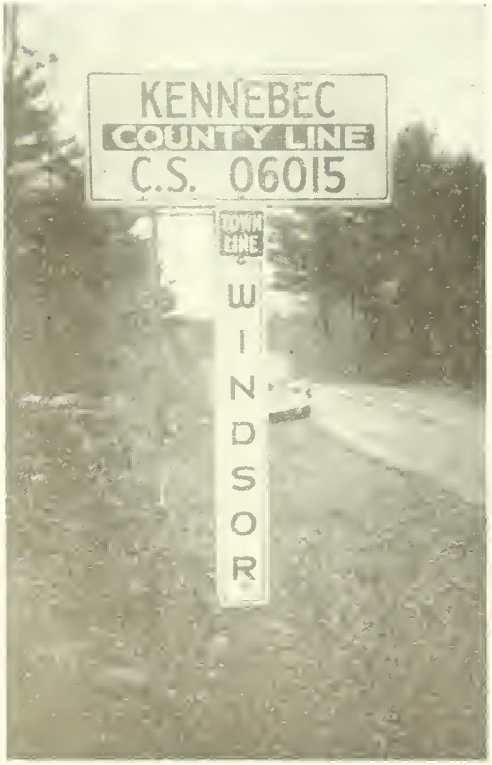

Figure 4.—A Maine control section marker.

Subsections

The various divisions of the highway department may, for their own purposes, desire to break down control sections into smaller segments. To accomplish this, control sections can be divided into subsections at urban limits, changes in surface type, variation in widths and ages, and for such highway facilities as bridges, interchanges, traffic circles, and grade crossings. Subsections are also utilized for traffic or accident studies, soil condition surveys, or any special purpose or study that a State desires to undertake. The establishment of subsections is ordinarily left to the discretion of each division of the highway department. Accordingly, the subsections may not be uniform among the various divisions.

There is no specifically recommended system of numbering subsections. One system in use involves numbering subsections consecutively within a control section, utilizing a two- or three-digit number. If it is desired to identify particular types of highway facilities, then blocks of numbers can be used for unincorporated or incorporated areas, bridges, special types of intersections, traffic circles, etc.

Another system uses, as an identifying number for each subsection, the distance, in hundredths of a mile, that the subsection is from the western or southern end of the control section. It may be preferable to ignore the actual direction of the individual control section and use the prevailing direction of the route on which the control section is located. In this manner, all subsections will be numbered according to mileages measured in the same general direction for any particular route, thus facilitating the preparation of logs, etc. If this scheme is used the first subsection within each control section will be numbered 0000, indicating that the subsection starts at one end of the control section. A subsection numbered 0351 would be the subsection starting 3.51 miles from the western or southern end of the control section.

Uniform Cost Accounting

From the standpoint of keeping cost records, only part of the job has been accomplished when control sections have been established. It is equally important that the costs themselves be reported on a uniform, logical, and practical basis.

The adoption of a uniform system of accounts by State highway departments will enable comparisons of costs and accomplishments among States to be made which would lead to more effective ways and means of planning, programing, constructing, and maintaining highways. Similar advantages can be obtained by local governmental units having responsibilities for road affairs.[4]

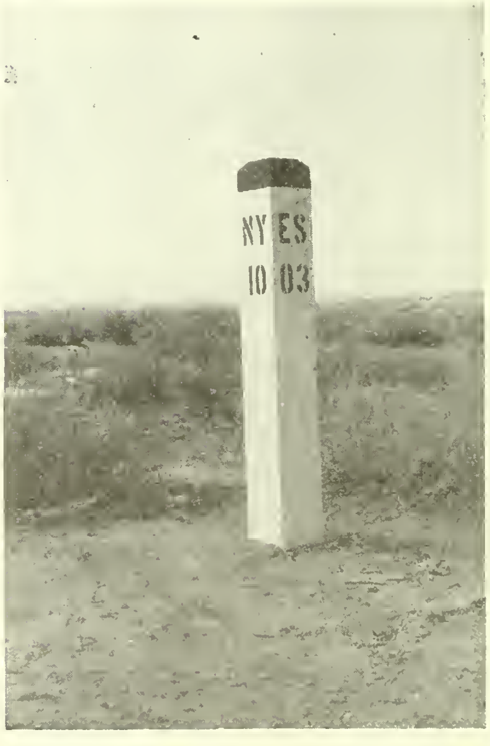

Figure 5.—A Nevada control section marker.

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[4] *Is cost accounting worth its cost [to counties]?* Better Roads, vol. 20, No. 9, September 1950. *The uniform cost accounting system for Wisconsin county highway departments*, by W. B. Blair, Western Builder, June 1, 1950. *A standardized system of fiscal and cost accounting* (manual for the office of the county engineer in the counties of Kansas), by a Committee of the Kansas County Engineers Association, August 1950.

The construction and maintenance cost data reported by control sections are being used by the officials of the State highway departments in planning financial, construction, and maintenance programs. One of the more recent uses being made of the construction and maintenance cost data is the determination of the annual cost of sections of the highway. Such work is currently under way in connection with the special load-condition surveys.

A practical and efficient system of cost accounting has long been recognized as an essential part of the routine functions of a highway agency. The generally accepted guide on accounting procedures is the preliminary manual of accounting procedure for State highway departments prepared by the Subcommittee on Uniform Accounting of the American Association of State Highway Officials. In a 1949 survey [5] of the accounting procedures of the State highway departments it was found that there were only 10 States which kept construction cost accounts that conformed to or had only minor deviations from the recommended accounts of the Subcommittee. In 3 States the construction accounts had major deviations from the recommended accounts, and in 35 States no classified construction accounts were kept at all.

According to the survey, however, even in States which have established uniform construction and maintenance cost accounts there are cases where sound accounting practices are violated to an extent that impairs the value of the cost records obtained. One of the most frequent deviations from the recommended procedures is in the prorating of many items of expense.

One of the objectives of the uniform accounting system is to show the true expenditures for construction and maintenance, regardless of whether construction or maintenance funds are involved, whether the work is done by contract or day labor, or whether it is performed under the supervision of construction or maintenance divisions. In many States, additions and betterments to the highway system are done by maintenance forces and paid for from maintenance funds. Since additions and betterments are construction and not maintenance, in that they are capital investments which increase the value of the highway facilities, they should be reported as construction even though maintenance funds are used and maintenance personnel perform the work.

Construction Cost Accounts

Construction cost accounts have been recommended by the Subcommittee on Uniform Accounting of the American Association of State Highway Officials for recording construction operations, and have been adopted in 17 States in connection with the establishment of control sections. The recommended construction accounts are as follows:

1. *Right-of-way:*
   - Purchase cost.
   - Costs in lieu of purchase.
   - Damages.
   - Expenses.
   - Right-of-way fence costs.
   - Right-of-way marker costs.
2. *Preliminary engineering:*
   - Salaries.
   - Expenses.
   - Equipment rentals for field parties on reconnaissance.
   - Materials investigations.
   - Foundation tests.
   - Office engineering in connection with field surveys.
   - Plans and estimate preparations.
   - Supplies for field parties.
3. *Construction engineering:*
   - Salaries.
   - Expenses.
   - Supplies.
   - Equipment rentals for field engineering.
   - Inspection costs.
   - Office engineering expenses.
   - Materials testing and inspection.
   - Preparation of final reports.
   - Project and station markers.
4. *Roadway drainage and grading:*
   - Clearing and grubbing.
   - Excavation of earth and rock.
   - Fine grading and excavation of subgrade.
   - Overhaul.
   - Borrow.
   - Back sloping and finishing earth shoulders.
   - Purchase cost of earth fill for roadway when title to land remains with seller.
   - Grading for approaches and driveway entrances.
   - Widening of shoulders, fills, and grade line changes.
   - Moving buildings, structures, and fences when not a right-of-way consideration.
   - Selection of material for subgrade.
   - Frost boil prevention work.
   - Rolling and compacting fills.
   - Shoulder lift when required by resurfacing roadway.
   - Fills for bridge and overpass approaches.
   - Open drainage ditches.
   - Channel changes.
   - Tile and other subdrains, except beneath roadway surfaces.
   - Dikes and levees.
5. *Drainage and earthwork protective structures:*
   - Culverts of 20-foot span or less.
   - Farm entrance culverts.
   - Flumes and wash checks.
   - Storm sewers.
   - Retaining walls.
   - Riprap and revetments.
   - Jetties, cribbing, and rock fills.
   - Slope walls.
   - Cattle passes and cattle guards.
   - Slope drains, catch basins, and manholes.
   - Gutters when not integral with roadway surface.
6. *Roadway surface and base:*
   - All materials used for subgrade treatment, and for base or roadway surface.
   - All materials used for curb and gutters when integral with surface and base.
   - Additions of bituminous seal coats, surface treatments, mats, or retreads which add ¾ inch or more in thickness in one operation.
7. *Improved shoulders and approach surfacing:*
   - Surfacing of shoulders, side approaches, and turnouts where type of material used is superior to adjacent roadside and inferior to roadway surface.
8. *Bridges, viaducts, grade separation structures, and tunnels:*
   - Structures over 20-foot span.
   - Excavation for above.
   - Excavation for approach structures (earth fill approaches included under roadway).
   - Concrete, steel, and wood structural work.
   - Protective structures of riprap.
   - Breakwaters, cribbing, retaining walls, and pier protection.
   - Roadway paving and sidewalks when integral with structure.
9. *Traffic and pedestrian service, and miscellaneous facilities:*
   - Guard fences.
   - Guide posts.
   - Permanent snow fences (structures or plantings).
   - Route and direction markers.
   - Signs.
   - Safety and signal devices.
   - Railroad crossing signals.
   - Highway lighting.
   - Pedestrian underpasses and overheads.
   - Sidewalks.
   - Permanent traffic lane stripes.
10. *Roadside development:*
    - Initial seeding, sodding, and planting of shoulders, back slopes, and roadside for the purpose of landscape and for protection of earthwork.
    - Bridle paths.
    - Roadside parks.
    - Picnic grounds.
    - Shelter houses.
    - Diversion ditches for protection of plantings.

Costs of adjustment of utility and other property, and detour expense during construction, where these items are carried separately in the contract pay items or where the work is done by day labor, may be classified separately if the State so desires, or they may be allocated to whichever of the construction cost classifications they are applicable.

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[5] *Status of fixed asset and maintenance cost accounting by State highway departments*, by the Committee on Highway Costs, Department of Economics, Finance, and Administration, Highway Research Board. Highway Research Correlation Service Circular No. 76, August 1949.

Reporting total construction costs of a completed project or addition and betterment job by these construction classifications will necessarily require a recapitulation of the items in each contract. This can be most satisfactorily accomplished by requiring that preliminary estimates, contractor's estimates, final vouchers, etc., show the groupings of the individual items into major classifications.

Maintenance Cost Accounts

The following maintenance cost accounts, recommended by the Subcommittee on Uniform Accounting, are in operation in the majority of the State highway departments:

1. *Routine roadway surface operations:*
    Patching holes and rough spots.
    Patching ruts and blow-ups.
    Repair of raveled edges.
    Sanding bleeding spots.
    Spot sealing.
    Temporary traffic ways.
    Dragging.
    Blading.
    Reshaping.
    Scarifying.
    Cleaning and raking.
    Picking up oversize rock.
    Filling and trimming expansion joints and cracks.
2. *Special roadway surface operations:*
    Application of dust palliatives when done annually or when the application does not result in a permanent improvement.
    Replacement of sand, sand-gravel, gravel, crushed stone, etc., on the same or a similar type of surface.
    Reprocessing or reconditioning bituminous surfaces or shoulders when little or no new materials are added.
    Bituminous surface treatment.
    Seal coating, when applied later than the following construction season.
    Light road-mixing operations.
    Major patching by special crews.
    Mud-jacking.
    Protection and handling of traffic during above operations.
    Addition of mats or retreads less than ¾-inch thickness.
3. *Shoulders and side approaches:*
    Patching.
    Dragging.
    Blading.
    Filling ruts.
    Replacing washouts.
    Reseeding or resodding, including original seeding on old work.
    Ribbon bituminous treatment (without excavation).
    Second or subsequent bituminous treatment.
    Replacement of gravel or stone.
4. *Roadside and drainage:*
    Repairing cuts, fills, slopes, and washouts.
    Removal of minor slides.
    Cleaning or retrenching drains, channels, and culverts.
    Maintaining drainage structures (20 feet or less).
    Removal and burning of weeds.
    Removal of debris.
    Planting or removal and trimming of trees.
    Brush removal.
    Planting or removal of shrubs.
    Seeding and sodding to prevent erosion.
    Care and replacement of special roadside development projects.
    Repair and maintenance of sidewalks, dikes, riprap, retaining walls, pumping stations, slope pavement, right-of-way fences.
5. *Traffic services:*
    Repair, repainting, and resetting of direction markers, route markers, signals, and gates, and the other safety devices.
    Magnetic dragging to remove iron.
    Traffic lane and guide line painting.
    Repair and repainting of guard rails.
    Highway lighting.
    Electricity for the operation of signals.
    Operation of comfort stations and picnic grounds.
    Detours not chargeable to construction or other maintenance operations.
6. *Snow, ice, and sand control:*
    Erection and removal of snow fence.
    Removal of snow and ice.
    Sanding icy surfaces.
    Snow and ice removal to open waterways.
    Removal of sand drifts.
7. *Structures (more than 20 feet in length):*
    Repair, maintenance, and operation of:
        Bridges.
        Tunnels.
        Subways.
        Overhead grade separations.
        Other structures.
    Operating expense of ferries.
8. *Extraordinary maintenance:*
    Special repairs and maintenance due to flood, storm, fire, major landslides or other catastrophe on:
        Roadway surface.
        Roadbed.
        Shoulders.
        Roadside.
        Drainage facilities.
        Safety devices.
        Structures.
    Handling and protecting traffic during emergency.
9. *Maintenance general expense:*[6]
    Prorata of district office and/or headquarters office expense chargeable to maintenance.
    Field maintenance supervision.
    Purchase and repair of small tools.
    Rental charges on inactive equipment.

Maintenance costs should be further broken down into objects of expenditure. The following objects of expenditure are suggested by the Subcommittee on Uniform Accounting: (1) Materials, supplies, and services; (2) salaries and wages; and (3) equipment operation. A further breakdown of these objects will depend on the requirements or desires of each State highway department.

Problems Encountered

The establishment of control sections and the adoption of uniform cost accounting procedures have posed many problems in the States. There are no standard guides to follow in solving these problems; thus far they have been handled individually in each State. However, so that the cost information from States that have adopted uniform accounting and cost keeping procedures will be on a uniform basis, study should be given to the solution of these problems, and recommendations prepared for consideration and adoption. Some of the problems that have been encountered are detailed in the following paragraphs.

Equipment rental rates.—It is essential that the States which adopt control sections and uniform accounting have equipment rental rates so that proper charges can be made to the control sections on which work by State forces is performed. In some States rates are already established and in operation while in others it is necessary that they be computed. These rental rates usually include depreciation, repairs, fuel, oil, storage, shop overhead, and insurance. The application of the rates varies from State to State. Some include the travel time of the equipment to and from the job, while others charge only the actual time the equipment is in operation at the job site.

Travel time.—When labor is paid for time to and from the job and travel time during the day between jobs, the problem arises as to the proper procedure for charging out this travel time. Whether the charges should be made on a daily basis to control sections on which work was performed or held in suspense and distributed on a monthly basis must be determined. If the charges are made directly to control sections, those which are the greater distance from the garage or district office are penalized by excessive travel time.

Sign and marker costs.—Some State highway departments manufacture their own road signs and markers. Costs are kept in a suspense account and distributed on the basis of the number of signs and markers used in each control section. In other States the costs of the signs and markers are determined by the previous year's costs and the overruns or underruns (over or under a specified percentage) are corrected at the end of the year.

Labor, equipment, and materials charges.—The reliability of the distribution of maintenance cost data in large measure depends upon the accuracy with which labor, equipment, and materials are charged by the foremen or timekeepers to individual sections of road. Constant check-ups accompanied by simplification of burdensome or complicated cost keeping procedures should be incorporated as part of the administrative control exercised over maintenance operations.

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[6] Suspense account, prorated to other cost items during or at the end of the accounting period.

Federal-aid Route Renumbering

In connection with the establishment of control sections in recent years it has been the practice of most States to avail themselves of the opportunity to renumber the Federal-aid routes and redesignate the Federal-aid projects. When adapting the Federal-aid procedures to a control section set-up in a particular State, it is desirable that such changes as are necessary be accomplished with the minimum number of adjustments. The expansion of the Federal-aid systems in the past has been accomplished by chronologically numbering routes as they were added, with the result that there is no logical pattern of numbering.

To overcome this deficiency, many of the States have employed a numbering procedure which gives the approximate geographical location of a route within a State. In this procedure, continuous routes within or across the State are selected in accordance with present-day traffic patterns. They are then numbered from south to north and from west to east; with sufficient numbers being skipped to provide for expansions, as routes are added or modified, without losing the initial advantage of the geographical arrangement.

Route numbers 1 to 9 are usually reserved for the interstate system, 10 to 99 for the Federal-aid primary system, and 100 to 9,999 for the Federal-aid secondary system. This system of numbering, where the number of digits identifies the road system, will not be adequate for those States where the Federal-aid primary system routes are in excess of 99.

Federal-aid Project Redesignation

The Federal-aid highway system in the majority of States is composed of numerous short Federal-aid projects for which the project number does not indicate system or route identification. When redesignating the Federal-aid projects, the old Federal-aid projects are handled as complete units in order to preserve the integrity of the Federal-aid records that have accumulated during the past 30 years. To accomplish this objective, the Federal-aid system is divided into new Federal-aid route sections, varying in length up to 100 miles, which embrace one or more complete old Federal-aid projects. Federal-aid route sections are numbered from west to east for routes originating in the west and from south to north for routes originating in the south. Projects are numbered with the digits preceding the dash representing the route number and the digit following the dash representing the route section number. For example, the first improvement on route 1, route section 1 would be designated as 1-1(1) in which the number within parentheses represents the project agreement number.

For the Federal-aid secondary system, the routes are not divided into sections and the entire route becomes the project. For example, the first improvement on route 100 would be 100(1) in which the number within parentheses represents the project agreement number.

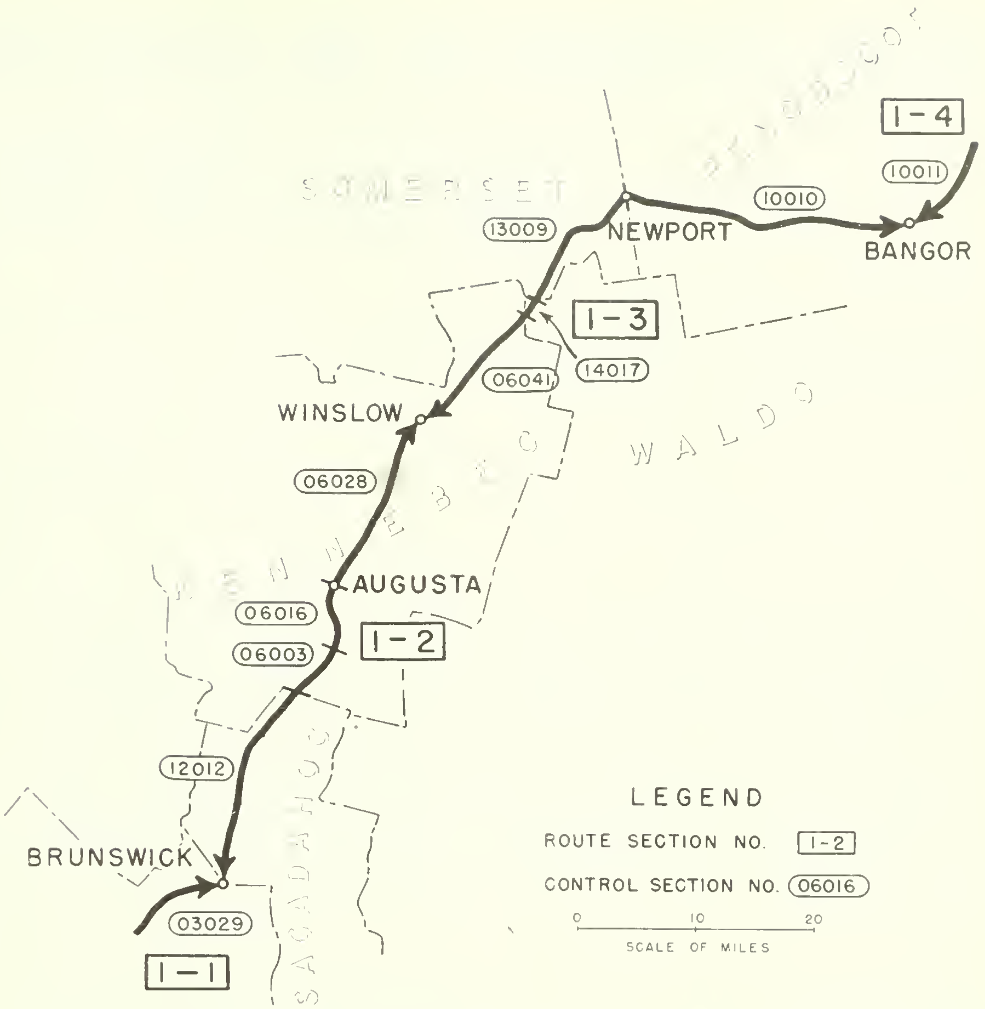

Figure 6.—Route sections and control sections on a portion of Maine Federal-aid route 1.

Figure 6 shows a section of Federal-aid route 1 in Maine that has been renumbered to conform with the new numbering procedure, divided into route sections (new Federal-aid projects) which contain a number of old Federal-aid projects in their entirety, and broken down into control sections for record keeping purposes.

Federal-aid route 1 is not to be confused with U S or State route 1. Federal-aid route numbering and U S or State route numbering are not necessarily colinear. For example, in figure 6, the portion of Federal-aid route 1 south of Brunswick is U S 1, from Brunswick to Winslow is U S 201, from Winslow to Newport is State routes 11 and 100, and from Newport to Bangor is U S 2.

Design Problems in the Use of Local Aggregates for Bituminous Surfaces

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported [1] by J. T. PAULS, Principal Highway Engineer
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Local aggregates often have to be used in constructing hot-mix bituminous road surfaces, sometimes in areas of severe or unusual climate. If the aggregates are substandard in quality or gradation, special design problems are involved which must be given careful attention. This article describes several studies of this nature.

Three points of timely interest relating to design are emphasized in these studies. First, the effectiveness of chemical additives in preventing water damage to hot asphalt mixtures has not been clearly established. Second, while crushing of rounded or weathered aggregates may increase resistance of the mixture to displacement in a dry condition, it may at the same time reduce the mixture's resistance to water action. Third, the behavior of mixtures containing both clay and limestone is unpredictable when subjected to water action.

BITUMINOUS road surfaces of the hot-mix type must often be constructed in areas where the use of local aggregates is imperative. In many cases, these local aggregates fail to meet recognized standards with respect either to quality or to gradation. Their successful use in bituminous mixtures, therefore, requires the solution of special design problems. These problems may be complicated when bituminous surfaces are to be built in regions where severe or unusual climatic conditions prevail.

It is the purpose of this report to describe briefly several examples of design studies of this nature. These examples illustrate some of the special considerations that must be taken into account when the contemplated construction involves the use of local aggregates having unusual properties, or when the bituminous roadway will be subjected to severe weather conditions. It is believed that a discussion of these studies will be of general interest to those who may encounter similar problems.

The data presented in this paper were abstracted from unpublished reports of laboratory studies of bituminous mixtures in which the use of local aggregates was an important factor. Each study was selected for inclusion by reason of some distinguishing feature requiring special consideration in design. Each also represents an area of differing climatic condition—southern Alaska, northern Alaska, and one of the Gulf States.

The first Alaska study was featured by the hydrophilic properties of most of the aggregates and the importance of this property in a region of heavy rainfall. Features of the second Alaska study were the unstable character of the natural aggregate and the severe weather conditions. The principal feature of the Gulf State study was the necessity to choose between using local aggregate in a relatively low cost sand-asphalt surface and using imported aggregate in a more expensive bituminous concrete.

Observations of Interest

The test results obtained in these studies either developed or emphasized three points relating to design that are of timely interest. First, the effectiveness of chemicals added to asphalt in preventing water damage to hot mixtures has not been clearly established. Second, although crushing of round and weathered particles of aggregate increases resistance of the mixture to displacement in a dry condition, it may have the effect of reducing the resistance of the mixture to water action, due to changes in surface characteristics of the particles other than angularity. Third, the behavior of mixtures containing both clay and limestone, when subjected to water action, is unpredictable. The behavior appears to be somewhat dependent on the kinds of clay and limestone involved.

In abridging the studies for inclusion in this report, it was thought desirable to present only those phases of design that assumed importance owing to singularities of the local conditions. For this reason the studies as presented should not be considered as being patterns of general design procedure. It is believed, however, that the information developed will be found of value in dealing with similar problems encountered in designing bituminous mixtures around the use of other local aggregates.

Study 1.—SOUTHERN ALASKA

In this study, investigating local aggregates from a number of sources in southern Alaska, it was desired to obtain information relating to the use of these aggregates in bituminous mixtures, with particular reference to resistance to the action of water. In case mixtures containing these aggregates were found to be unsatisfactory in this respect, it was desired to determine whether improvement could be made by using asphalt treated with a chemical additive. This aspect of design was of essential importance because of the extremely wet climate of the region, the annual rainfall being about 90 inches.

Although aggregates from 13 sources were included in the study, information on only one will be described here, in the interest of brevity. The methods used in testing this particular aggregate will serve as an example of the general procedures used with all the materials in the study. Several special mixtures containing this aggregate were prepared, and the results of the tests on these mixtures are of particular interest.

Petrographic analysis showed this material to consist of rounded and subangular schist, granite, gneiss, quartz, and granodiorite. Its apparent specific gravity was 2.74. Prior to preparing bituminous mixtures, the aggregate was separated on a

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[1] Paper presented at the 1949 annual meeting of the Association of Asphalt Paving Technologists.

Table 1.—*Gradation of aggregate, study 1*

Passing sieve size—	Material as received	Material as used
	Percent	*Percent*
1½-inch	80	
1-inch	72	
¾-inch	66	100
½-inch	58	88
⅜-inch	54	80
No. 4	42	64
No. 10	31	46
No. 40	8	12
No. 80	1.4	2
No. 200	.4	1

¾-inch sieve and the oversize material rejected. The gradings of the sample as received in the laboratory and of the material as used in the mixtures are shown in table 1.

Three groups of mixtures were then prepared, using 85–100 penetration asphalt. The first group contained untreated asphalt. The others contained asphalt treated with a chemical additive in the proportion of 1 and 2 percent by weight of the asphalt. In all these mixtures 4.5 parts of bitumen to 100 parts of aggregate, by weight, were used. Cylindrical specimens 4 inches in diameter and 4 inches high were molded from each mixture, and immersion-compression tests made. The procedure used in preparing the mixtures and testing the specimens was that described in PUBLIC ROADS in 1948 [2]. The test results are shown in table 2 (mixtures 1–3).

Results of Tests

The results of tests on the three groups of mixtures showed compressive strengths for the dry specimens that, while not high, would be considered adequate. Tests on the specimens after immersion showed that the mixture containing untreated asphalt had been damaged considerably in the soaking process. Although the stability-retention values of the mixtures containing treated asphalt were higher than those containing untreated asphalt, the results showed that the additive in either proportion used was not highly effective in preventing damage by water action. It therefore seemed desirable to determine whether or not resistance to the action of water could be improved by means other than treating the asphalt.

Former work had indicated that the stability-retention of mixtures may often be improved by incorporating a hydrophobic filler. Accordingly, another mixture was prepared using untreated asphalt. In the prior untreated-asphalt mixture the only filler had been the 1 percent of natural dust. In the second mixture (No. 4 in table 2) limestone dust was added to the aggregate in an amount sufficient to produce a combined aggregate having 6 percent passing the No. 200 sieve. This increase in filler permitted the use of 5 percent asphalt. In preparing the mixture for the last specimen of the usual set of six, 0.5 percent hydrated lime was added to the batch after the asphalt had been added. Immersion-compression tests were then made on these specimens, with the results also shown in table 2 (mixtures 4 and 4A).

Comparing the results of the limestone-dust mixture (No. 4) with those of the corresponding mixture (No. 1) containing the small amount of natural dust, the improvement due to the addition of the limestone dust was striking, with respect both to the compressive strength of the dry specimens and to the retained strength after immersion. Also, although the test value obtained on only one specimen cannot be considered as more than indicative, the considerably higher compressive strength after immersion of the single specimen containing lime hydrate, as compared with the average of two corresponding specimens that did not contain this material, suggested that mixtures containing this aggregate would be substantially improved by incorporating hydrated lime in the proportion used in the test mixture.

Conclusions Drawn

The test results led to the following conclusions as they apply to the one aggregate discussed here:

1. Tests on the dry mixtures showed compressive strengths that would be considered adequate for the expected traffic.

2. Immersion-compression tests showed that mixtures containing this aggregate would require some kind of special treatment if they were to be used successfully under unfavorable moisture conditions.

3. The one additive used in these tests was not highly effective in preventing damage due to water action. Some additives appear to be more effective than others when used with certain aggregates, and it is possible that additives other than the one used in these tests might be more satisfactory with this aggregate. Also, it has not been clearly determined whether or not the effect of chemical additives is changed by the elevated temperatures required in preparing hot mixtures. It may be that these temperatures alter the properties of the additive and so reduce its effectiveness.

4. Resistance to the action of water of bituminous mixtures containing this aggregate could be improved by adding limestone dust as a filler. The addition of small proportions of hydrated lime might also be very effective.

Study 2.—NORTHERN ALASKA

This study concerned the design of a bituminous mixture for pavement construction in northern Alaska, in a region having much more severe weather conditions than the area involved in the Alaska job in study 1. The air temperature exceeds 32° F. for a very short period in summer only, and seldom exceeds 60° F. The entire area was overlaid to a considerable depth with coarse sand and fine gravel. At all times other than during the short summer season, this material was icebound, and surface stability was not a problem. During the brief warm period, however, the sand and fine gravel were loose and unstable to such a depth as to require some form of surface stabilization.

Table 2.—*Results of immersion-compression tests, study 1*

Mixture No.	Parts bitumen to 100 parts aggregate, by weight	Parts additive to 100 parts asphalt, by weight	Characteristics of specimens, as molded [1]			Characteristics of specimens after 4 days immersion at 120° F.[1]			
			Specific gravity	Air voids	Compressive strength, 77° F.	Absorption	Volume increase	Compressive strength, 77° F.	Retained strength [2]
				Percent	*Lb./sq. in.*	*Percent*	*Percent*	*Lb. sq. in.*	*Percent*
1	4.5	0	2.21	13.7	165	5.2	1.7	94	57
2	4.5	1	2.22	13.3	180	5.5	1.7	120	67
3	4.5	2	2.22	13.3	174	5.7	1.7	124	71
4 [3]	5.0	0	2.30	9.4	238	[4] 2.4	[4] 1.7	[4] 197	83
4A	5.0	0	[5] 2.31	9.1		2.2	1.2	[5] 238	[6] 100

[1] Except as noted, values are the average for three specimens.
[2] Retained strength = (compressive strength of immersed specimens ÷ compressive strength of dry specimens) × 100.
[3] Mixture contained 5 percent added limestone dust.
[4] Average value for two specimens.
[5] Same as mixture 4, except that 0.5 percent hydrated lime was added following addition of the bitumen. Only one specimen was molded, due to insufficient material.
[6] Based on compressive strength of mixture 4 as molded.

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[2] *Further developments and application of the immersion-compression test*, by J. T. Pauls and J. F. Goode. PUBLIC ROADS. vol. 25, No. 6, December 1948.

Table 3.—*Gradation of aggregate, study 2*

Passing sieve size	Original material	Material after crushing and recombining
	Percent	*Percent*
$^3/_8$-inch	100	100
No. 4	87	98
No. 8	38	68
No. 10	30	56
No. 20	9	20
No. 30	7	14
No. 40	5	11
No. 50	4	7
No. 80	1.2	3.4
No. 100	.8	2.6
No. 200	.6	1.6

The aggregate consisted of predominately subangular fragments of dark, fine-grained volcanic rock, rhyolite and andesite, quartz and some quartzite. Its apparent specific gravity was 2.65, and absorption 0.9 percent. Stripping tests were made on the ⅜-inch to No. 4 sieve fraction of the aggregate, using both the Nicholson and static immersion methods. Two grades of asphalt were used: 85–100 penetration, and 200–300 penetration. Results of both test methods with both grades of asphalt showed 100-percent retention of bituminous coating. The gradation of the natural aggregate is shown in the second column of table 3.

In order to determine its compactibility, vibratory compaction tests of the aggregate were made. To investigate the possibility of improving density by altering the gradation, three samples were prepared for test. One sample consisted of the natural aggregate. In the second sample the percentage of material passing the No. 10 sieve was increased to 40 percent by sieving and recombining. In the third sample, only material passing the No. 10 sieve was used. The vibratory density test results of the three materials showed air voids of 26.6 percent in the material of unaltered gradation, 25.4 percent in the material containing 40 percent passing No. 10 sieve, and 25.4 percent in the material all passing No. 10 sieve.

Trial Mixtures

A limited number of trial mixtures of asphalt and aggregate were then prepared. The objectives of the preliminary work were, first, the selection of an appropriate grade and proportion of asphalt, and second, some indication of the stability to be expected from mixtures containing the natural aggregate and from mixtures containing aggregate altered by manipulating the gradation or by other means.

In consideration of the severe moisture and temperature conditions at the job site, the use of a penetration grade asphalt seemed desirable. Moreover, since the natural aggregate was lacking in mechanical stability due to smoothness and roundness of the particles, it was decided to include in the trial mixtures a 40–50 penetration asphalt as well as an 85–100 asphalt. Four aggregate combinations were used in the trial mixtures: (1) the natural aggregate, (2) aggregate prepared from the native material by sieving and recombining so that 40 percent passed the No. 10 sieve, (3) aggregate prepared from the native material by sieving so that 50 percent passed the No. 10 sieve, and (4) aggregate prepared by crushing the 62 percent naturally retained on the No. 8 sieve and recombining the product of the crusher with 38 percent originally passing the No. 8 sieve. Crushing the larger size fractions, of course, resulted in marked alteration of particle shape. The change in gradation is shown in the third column of table 3.

Cylindrical test specimens, 4 inches in diameter and 4 inches high, were molded from the trial mixtures and tested in compression at two temperatures: 60° F. and 77° F. The former temperature was selected by reason of its being the normal maximum air temperature at the job site, and the latter was used in order to correlate the test values in this study with those obtained in other work.

To conserve material, the 85–100 penetration asphalt specimens, after having been tested at 77° F., were reheated and remolded for testing at 60° F. The specimens containing 40–50 penetration asphalt were first tested at 60° F., then reheated, remolded, and tested at 77° F. Although the strength of recompressed specimens is usually higher than that of original ones, sufficiently clear indications of the effect of the variables in the preliminary work were shown in the test results, which are given in table 4.

The results of these preliminary tests led to the following observations:

1. Specimens containing 85–100 penetration asphalt and the natural aggregate produced compressive strengths at either test temperature that would be considered inadequate for the purpose intended.
2. Use of 40–50 penetration asphalt with natural aggregate produced considerably higher stability than was obtained with 85–100 asphalt.
3. Alteration of aggregate gradation by separation and recombination failed to provide sufficient stability.
4. Alteration of aggregate gradation and particle shape by crushing resulted in pronounced improvement in stability.

Final Mixture Types

Based on the results of the preliminary tests three types of mixtures were prepared. One was composed of the natural aggregate and 5 percent asphalt; the second was composed of aggregate altered by crushing and recombining, and 5 percent asphalt; and the third was composed of the crushed and recombined aggregate and 6 percent asphalt. The grade of asphalt used in these mixtures was 40–50 penetration. Prior to mix-

Table 4.—*Results of tests on preliminary specimens, study 2*

Composition of specimen	Proportion of asphalt	Compressive strength—[1]	
		at 60° F.	at 77° F.
	Percent	*P.s.i.*	*P.s.i.*
Group 1: with 88 penetration asphalt and			
Natural aggregate	3	126	32
Do	4	126	76
Do	5	120	36
Altered aggregate, 40 percent passing No. 10 sieve	4	138	48
Altered aggregate, 50 percent passing No. 10 sieve	4	137	43
Group 2: with 47 penetration asphalt and			
Natural aggregate	4	205	71
Do	4½	236	86
Do	5	249	86
Altered aggregate, with 62 percent crushed aggregate	5	302	117

[1] In group 1, the tests at 77° F. were made first, and the specimens were then reheated and remolded for testing at 60° F. In group 2, the tests at 60° F. were made first, and the specimens were then reheated and remolded for testing at 77° F

Table 5.—*Final tests on specimens containing local aggregate and 40-50 penetration asphalt, study 2*

Composition of specimens	After 7 days immersion at 77° F.		After 7 cycles of freezing and thawing		Compressive strength at—		Retained stability [1]
	Absorption	Swell	Absorption	Swell	60° F.	77° F.	
	Percent	*Percent*	*Percent*	*Percent*	*P.s.i.*	*P.s.i.*	*Percent*
Natural aggregate, with 5 percent asphalt	(2)	(2)			191	94	
	5.2	1.3				86	91
			4.9	3.4		82	87
Aggregate altered by crushing, with 5 percent asphalt	(2)	(2)			230	122	
	7.3	0				98	80
			6.4	.2		101	83
Aggregate altered by crushing, with 6 percent asphalt	(2)	(2)			295	156	
	7.4	0				121	78
			6.0	.1		129	83

[1] Retained stability = (compressive strength of specimens after conditioning at 77° F. ÷ compressive strength of dry specimens at 77° F.) × 100.
[2] No immersion.

ing, the aggregate was heated to 320° F. and the asphalt to 300° F. Mixing was done in a kitchen-type mixer. The temperature of the mixtures at molding was 225° F. The molding load was 3,000 pounds per square inch, held for 2 minutes. Eight specimens of each mixture were prepared. Of these, four were tested in compression at 60° F. and 77° F. without treatment other than overnight storage at laboratory temperature. The other four specimens were placed in a vacuum bath to accelerate saturation of the specimens by water. The specimens were covered with water and vacuum applied for 20 minutes, the degree of vacuum used being 27 inches of mercury. Following this operation, two of the specimens were placed in a water bath at 77° F. for 7 days. The other two specimens were subjected to seven freezing and thawing cycles, each cycle consisting of 4 hours exposure to a temperature of −10° F. and 20 hours immersion in water at 77° F. These specimens were also tested for compressive strength at 77° F. Prior to testing, measurements were made of the volume change and absorption of the specimens that had been subjected to immersion and to freezing and thawing. Results of the principal series of tests are shown in table 5, and led to the following observations:

Comparing the compressive strengths of the two groups of dry specimens containing 5 percent asphalt, it is seen that the use of crushed aggregate resulted in substantial increases in stability at both test temperatures.

Immersion in water at 77° F. produced little stability reduction in the natural aggregate specimens. The somewhat lower stability retention noted in corresponding specimens containing crushed aggregate might be explained either by the higher percentage of water absorption or by the possibility that the bitumen coatings on freshly crushed particles may be more susceptible to water action than those on the surface of weathered particles of the same aggregate. Incidentally, since this study was made, the use of the vacuum bath has been dropped from the immersion-compression test procedure because it produced high water absorption in mixtures that otherwise were resistant to water permeation.

Alternate freezing and thawing resulted in little loss of stability in the natural aggregate mixture, and the crushed aggregate mixture showed slightly lower retention of strength. The stability losses due to immersion and to freezing and thawing were essentially the same for both crushed aggregate mixtures.

Recommendations Made

These observations led to the following recommendations for designing a bituminous mixture for the particular conditions of the job:

1. The aggregate should be altered by crushing the portion retained on the No. 8 sieve, and recombining the crushed material with the portion passing the No. 8 sieve.
2. A hot mixture should be used.
3. The asphalt used should be one of the lower penetration grades, such as 40–50 penetration.
4. The optimum percentage of asphalt for density and workability will probably fall within a range of 5.5 to 6.5 parts of asphalt to 100 parts of aggregate by weight. In consideration of prevailing temperature and moisture conditions the use of a rich mixture within these limits, consistent with workability, would be desirable.

Study 3.—A GULF STATE

This study for the design of a bituminous surfacing for a section of park roadway in one of the Gulf States, is presented by way of contrast to the other two studies, where special considerations were required because of a short working season, low temperatures, high rainfall, and the necessity to use the aggregate found at hand. All the conditions were different in the parkway design problem. The climate is mild, the working season long, rainfall is moderate, and a choice was possible between local aggregates and commercially produced aggregates. The problem was to determine whether or not a satisfactory sand-asphalt surface course could be made, using sand from pits near the roadway. If this were not possible, it was required to design a bituminous concrete mixture containing sand and crushed gravel from one of two commercial sources. The sand-asphalt type would, of course, be much less expensive.

The sand-asphalt study was limited to the use of 85–100 penetration asphalt and emulsified asphalt of the mixing type. Use of 85–100 penetration asphalt only was desired in the design of the bituminous concrete mixtures.

Sieve analysis, specific gravity and absorption determinations, and petrographic analysis were made on all the aggregates. Results of these tests are given in tables 6 and 7. Results of tests to determine the clay content and plastic properties of the three pit sands, S-4, S-5, S-6, are shown in table 8. It was apparent from visual examination that the sands from the commercial sources were free from clay.

Two of the local sands, S-4 and S-5, were from different parts of the same pit, and in order to determine the advantage of blending them, vibratory density tests were made on several combinations with the results shown in table 9.

Sand-Asphalt Mixtures

Sand-asphalt mixtures were prepared for Hubbard-Field stability and immersion-compression tests. Since the vibratory tests failed to show any advantage in blending sands S-4 and S-5, only the one that showed the higher density, S-5, was included in these mixtures. The test characteristics of the two bituminous materials used follow.

Table 6.—*Results of petrographic analysis and specific gravity and absorption determinations, study 3*

Kind of material	Field sample No.	Apparent specific gravity	Absorption	Petrographic analysis
			Percent	
Commercial sand	S-1	2.66	0.7	Quartz and chert.
Do	S-3	2.66	.3	Angular and subangular fragments of quartz with some chert and sandstone.
Pit sand	S-4	2.66	1.4	Angular fragments of quartz.
Do	S-5	2.65	1.6	Angular fragments of quartz, with some feldspar.
Do	S-6	2.66	2.7	Do.
Commercial gravel	G-1	2.61	3.2	Chert with some quartz.
Do	G-2	2.63	3.7	Chert with some quartz and sandstone.
Do	G-5	2.62	2.5	Angular and subangular fragments of chert with some quartz.
Do	G-6	2.63	2.7	Angular and subangular fragments of chert and quartz.

Table 7.—*Gradation of aggregate, study 3*

Percentage of material passing sieve size	Commercial sand		Pit sand			Commercial gravel			
	S-1	S-3	S-4	S-5	S-6	G-1	G-2	G-5	G-6
1½-inch						100		100	
1-inch						93		96	
¾-inch						82	100	83	
½-inch						55	99	44	
⅜-inch	100	100				37	94	21	100
No. 4	99	96				9	32	1	24
No. 10	96	87	100		100		4		1
No. 20	90	75	98	100	98				
No. 30	81	64	94	98	93				
No. 40	62	46	83	91	73				
No. 50	31	22	60	63	45				
No. 80	7	5	18	29	18				
No. 100	4	3	11	23	15				
No. 200	1	1	6	14	12				

Table 8.—*Clay content and plastic properties of pit sands, study 3*

Sample No.	Hydrometer analysis of material passing No. 10 sieve			Physical constants of material passing No. 40 sieve	
	Sand	Silt	Clay	Liquid limit	Plasticity index
	Percent	*Percent*	*Percent*		
S-4	92	1	7	NP[1]	NP
S-5	86	3	11	NP	NP
S-6	86	4	10	17	NP

[1] NP = not plastic.

Table 9.—*Results of vibratory density tests on blends of local sands, study 3*

Composition of blend		Solids after vibration
Sand S-4	Sand S-5	
Percent	*Percent*	*Percent*
100	0	66.7
80	20	67.4
60	40	67.8
40	60	68.4
20	80	69.4
0	100	69.3

Emulsified asphalt:	
Specific gravity, 77°/77° F.	1.013
Viscosity, Saybolt Furol, 77° F.	126
Demulsibility, 50 ml. 0.1 N. CaCl₂ soluble, percent	0
Sieve test, percent retained	0
Settlement test, percent (creaming) difference	−0.8
Stone coating test	Pass
Mixing test, percent	Broken 1.0
Dehydration test, loss, percent	80.5
Miscibility test	Pass
Distillation test, residue, percent	62
Test on residue:	
Penetration, 77° F., 100 gm, 5 sec.	186
Ductility, 77° F., cm.	72
Total bitumen (soluble in CS_2), percent	98.02
Organic matter insoluble, percent	1.08
Inorganic matter insoluble, percent	0.90
Ash by ignition, percent	1.12
Specific gravity, 77°/77° F.	1.018
85-100 penetration asphalt:	
Specific gravity, 77°/77° F.	1.027
Flash point, ° F.	520
Softening point, ° F.	115.2
Penetration, 77° F., 100 gm., 5 sec.	93
Ductility 77° F., 5 cm./min., cm.	180
Loss, 163° C., 5 hours, percent	0.12
Penetration of residue	85
Total bitumen (CS_2)	99.99
Organic matter insoluble, percent	0.01
Oliensis test	Negative

In the hot mixtures, two proportions of asphalt were used with each of the sands S-5 and S-6: 9 and 10 parts of aggregate, by weight. In the cold mixtures two proportions of emulsified asphalt were used with each sand: 7 and 9 parts of water-free bitumen to 100 parts of air-dry aggregate, by weight. For each sand and with each type and proportion of bituminous material, two types of aggregate were used, one consisting of the sand alone and the other composed of a combination of sand and limestone dust.

Mixing was done in a kitchen-type mixer. For the hot mixtures the mixing temperatures of aggregate and asphalt were 325° F. and 300° F., respectively. For the cold mixtures, each batch of aggregate was moistened with 3 percent of water prior to adding the emulsified asphalt, to facilitate dispersion of the emulsion.

Two types of test specimens were molded from each batch. Hubbard-Field test specimens were 2 inches in diameter and 1 inch in height. Specimens for the immersion-compression test were 2 inches in diameter and 2 inches high. Stability test specimens were made in sets of three, and immersion-compression specimens in sets of six. Hot mixture specimens were molded immediately after mixing. Cold mixtures were cured in a loose condition for 18 hours after mixing in a 140° F. oven. The temperature of hot mixtures at the time of molding was 260° F., and that of the cold mixtures was 140° F. All specimens were compacted under a static load of 3,000 pounds per square inch. They were cured in the 140° F. oven for 24 hours after molding, after which period they were allowed to cool to room temperature for density and volume measurement.

The Hubbard-Field specimens were tested at a temperature of 140° F. The results are given in table 10. For the immersion-compression tests the immersion water temperature was 120° F., and the test temperature was 77° F. The immersion-compression test results of the sand-asphalt mixtures are also shown in table 10.

In general, the Hubbard-Field stability values obtained for mixtures containing sand from either of the two local pits, with either type of bituminous material, would be considered satisfactory. Mixtures containing either of the sands and the lower proportion of emulsified asphalt appeared dry, with many poorly-coated particles or agglomerations. Use of the higher proportion of emulsified asphalt resulted in improved appearance, although the mixtures still would be considered to be lean.

Table 10.—*Hubbard-Field stability tests and immersion-compression tests on sand-asphalt mixtures, study 3*

Composition of specimen	Parts limestone dust to 100 parts sand, by weight	Parts bitumen to 100 parts aggregate, by weight	Hubbard-Field stability test			Immersion-compression test						
			Characteristics of specimens as molded		Hubbard-Field stability at 140° F.	Characteristics of specimens as molded			Characteristics of specimens after 4 days immersion at 120° F.			
			Air voids	Voids in aggregate		Air voids	Voids in aggregate	Compressive strength	Absorption	Swell	Compressive strength	Retained strength
			Percent	*Percent*	*Lb.*	*Percent*	*Percent*	*Lb./sq. in.*	*Percent*	*Percent*	*Lb./sq. in.*	*Percent*
Sand S-5 with 85-100 penetration asphalt	0	9	18.5	34.2	850	19.3	34.8	181	7.3	5.7	52	29
	0	10	17.2	34.2	870	17.9	34.8	208	6.6	5.4	71	34
	6	9	16.2	32.0	1,260	16.9	32.6	272	7.5	8.1	58	21
	6	10	14.2	31.8	1,310	14.8	32.2	288	6.3	7.4	86	30
Sand S-5 with emulsion asphalt	0	7	21.4	33.5	1,340	21.3	33.5	152	7.0	10.9	13	9
	0	9	17.5	33.2	1,470	17.1	32.9	204	5.1	6.8	29	14
	6	7	20.9	33.1	1,340	20.4	32.7	169	(1)	(1)	(1)	0
	6	9	16.1	32.0	1,620	15.7	31.8	239	7.5	9.6	14	6
Sand S-6 with 85-100 penetration asphalt	0	9	18.7	34.1	1,170	19.4	34.6	233	7.4	5.2	69	30
	0	10	17.8	34.7	1,150	17.3	34.3	253	6.0	4.9	102	40
	6	9	17.4	33.0	1,350	16.4	32.3	297	7.9	8.9	64	22
	6	10	15.5	33.0	1,300	14.7	32.3	313	6.9	8.1	91	29
Sand S-6 with emulsion asphalt	0	7	20.6	33.0	1,200	20.8	33.1	156	9.6	6.0	15	10
	0	9	16.5	32.4	1,320	16.5	32.5	199	6.6	3.9	32	16
	6	7	18.9	31.4	1,300	18.8	31.4	192	[2] 14.1	[2] 11.9	[2] 8	[2] 4
	6	9	14.2	30.6	1,490	14.3	30.7	249	8.1	6.2	21	8

[1] These specimens cracked and failed after 3 hours immersion.
[2] These specimens cracked during the immersion period.

Observations Made

The immersion-compression tests indicated that sand-asphalt mixtures containing sand from either pit would be unsatisfactory in resistance to loss of stability due to the action of water. From the test data, the following observations are of interest:

1. Mixtures containing either of the sands and the penetration grade asphalt showed better resistance to water action than those containing the same sand and the emulsion in corresponding proportions of bitumen. It is likely that the lower resistance of the emulsion mixtures was due to lack of complete and uniform coating provided by the proportions used.
2. With one exception, dry specimens of mixtures containing either of the sands and the penetration-grade asphalt had higher compressive strengths than those containing the same sand and the emulsion in corresponding proportions of bitumen.
3. With either sand, mixtures containing limestone dust showed greater stability loss due to immersion than corresponding plain sand mixtures. No explanation for this is clear at this time, although similar phenomena were observed in early work in developing the immersion-compression test. It may be that certain types of limestone are more reactive than others when combined with certain types of clay.
4. The percentage of air voids was high, even in the mixtures containing limestone dust and the higher proportion of each bituminous material. The advantage to be gained by reducing the air voids by additional amounts of asphalt and filler with these particular sands is extremely doubtful, in view of the very low values for retained strength obtained for all the mixtures in the immersion-compression test.

Bituminous Concrete Mixtures

The design of the alternate bituminous concrete mixtures presented no special difficulty. The results of Los Angeles abrasion tests made on the coarser gravels from the commercial sources (using A.A.S.H.O. designation grading A) showed abrasion loss of 15.1 percent for sample G-1 and 18.0 percent for sample G-5.

Static immersion tests made on the ⅜-inch to No. 4 fraction of the gravels gave the results shown in table 11.

From density tests made on molded specimens of a series of trial mixtures, the optimum proportions of coarse and fine aggregates were determined. From these trial mixtures a range of bitumen contents was also determined. Final mixtures were then prepared, and immersion-compression tests made, using the procedure followed in study 1. The results are shown in table 12. In this table, mixtures 1 and 2 are the wearing course and leveling course mixtures, respectively, containing sand and gravel from source *A*. Mixtures 3 and 4 are corresponding mixtures containing aggregates from source *B*. Although the compressive strengths of dry specimens containing aggregate from source *A* were higher than those of the specimens containing aggregate from source *B*, the results in both cases indicated sufficient stability for park roads on which traffic is not expected to be heavy. The immersion tests indicated satisfactory resistance to water action for all four mixtures.

As the result of this study, it was concluded that the use of sand-asphalt would not be recommended for this project, and that the use of the more expensive bituminous concrete would be justified.

Table 11.—*Static immersion test of gravels, study 3*

Sample	Area remaining coated after 24 hours immersion in water at		
	100° F.	120° F.	140° F.
	Percent	*Percent*	*Percent*
G-1	95	85	80
G-5	90	80	50

Table 12.—*Composition of bituminous concrete mixtures and results of immersion-compression tests, study 3*

	Mixture 1	Mixture 2	Mixture 3	Mixture 4
Coarse aggregate, sample No	G-1	G-2	G-5	G-6
Coarse aggregate, percent by weight	56	65	49	55
Fine aggregate, sample No	S-1	S-1	S-3	S-3
Fine aggregate, percent by weight	38	29	45	39
Limestone dust added, percent by weight	6	6	6	6
Gradation of composite aggregate, percentage passing—				
1-inch sieve	100		100	
¾-inch sieve	93	100	94	
½-inch sieve	77	99	74	
⅜-inch sieve	66	96	62	100
No. 4 sieve	48	55	50	57
No. 10 sieve	42	36	45	41
No. 20 sieve	40	32	40	35
No. 30 sieve	37	29	35	31
No. 40 sieve	30	24	27	24
No. 50 sieve	18	15	16	15
No. 80 sieve	9	8	8	8
No. 100 sieve	8	7	7	7
No. 200 sieve	7	6	6	6
Parts asphalt to 100 parts aggregate, by weight	6.0	6.0	5.3	5.5
Characteristics of specimens as molded:				
Air voids, percent	8.7	10.2	8.9	8.1
Voids in mineral aggregate, percent	21.0	22.2	19.4	19.5
Compressive strength, lb. per sq. in	184	181	151	155
Characteristics of specimens after 4 days immersion:				
Absorption, percent	2.2	2.3	2.0	1.7
Volume change (swell), percent	.4	.4	0	0
Compressive strength, lb. per sq. in	178	176	159	164
Retained strength, percent	97	97	100	100

Title Pages for Volumes 24 and 25

In years past the Bureau of Public Roads has issued an alphabetically arranged index for each volume of Public Roads, the last being that for volume 23 (March 1942-June 1944). The indexing was discontinued at that time for lack of personnel, and has not been resumed.

Searching through the long series of alphabetically arranged volume indexes is always time consuming and often unsatisfactory. It has therefore been decided to discontinue permanently the indexing of individual volumes. Instead, a far more useful reference work will be published, in the form of a cumulative index covering all articles in Public Roads volumes 1–26. This will probably include a chronological listing of articles, an alphabetical list of authors, and a classification of articles according to subject matter.

The cumulative index will probably be published some time in 1952, after the issuance of the last issue number of volume 26. Readers are requested to please withhold orders for the cumulative index until after announcement of its actual publication. Neither the Bureau or the Superintendent of Documents can handle advance orders.

Because there are many who regularly bind each volume of Public Roads, we will continue to print a title page for each volume, in which will appear a list of the articles and their authors. The title pages for volumes 24 and 25 are being mailed to all subscribers of Public Roads.

The title pages will also serve as useful interim supplements to the cumulative index. While no definite plans have as yet been made, it is probable that supplemental cumulative indexes will be published periodically in the future, perhaps at intervals of 5 or 6 years.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

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(See also adjacent column)

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(See also adjacent column)

Public Roads Administration Annual Reports:
1943. 1944. 1945.

MISCELLANEOUS PUBLICATIONS

Bibliography on Automobile Parking in the United States.
Bibliography on Highway Lighting.
Bibliography on Highway Safety.
Bibliography on Land Acquisition for Public Roads.
Bibliography on Roadside Control.
Express Highways in the United States: a Bibliography.
Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.

U.S. GOVERNMENT PRINTING OFFICE: 1951—911332

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF JUNE 30, 1951

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$15,739	$13,745	$7,010	360.9	$7,345	$3,714	162.7	$16,948	$8,208	408.5	$38,038	$18,932	932.1
Arizona	1,255	7,553	5,308	160.6	892	516	13.2	4,340	3,109	49.9	12,785	8,933	223.7
Arkansas	4,028	12,852	6,757	423.1	5,325	2,656	175.3	13,360	6,838	360.1	31,537	16,251	958.5
California	9,165	39,858	6,713	182.9	19,295	9,230	75.5	62,867	30,679	287.9	122,020	46,622	546.3
Colorado	3,586	4,345	2,455	151.1	4,507	2,495	121.5	13,357	7,315	273.1	22,209	12,265	545.7
Connecticut	3,236	6,247	3,174	17.9	2,805	1,588	7.2	8,637	4,618	8.1	17,689	9,380	33.2
Delaware	2,774	1,004	599	22.5	10	9		5,235	2,553	19.1	6,249	3,161	41.6
Florida	4,719	12,932	6,615	292.6	7,751	4,075	117.2	19,897	9,787	417.9	40,580	20,477	827.7
Georgia	8,165	11,487	5,896	142.2	9,722	3,977	68.3	31,873	15,970	680.3	53,082	25,843	890.8
Idaho	5,431	8,414	5,309	339.4	2,308	1,416	87.9	8,115	4,580	233.5	18,837	11,305	660.8
Illinois	21,889	34,192	18,635	324.3	19,238	9,688	212.0	68,177	35,139	550.0	121,607	63,462	1,086.3
Indiana	11,887	37,216	18,633	222.6	8,869	4,306	181.9	16,955	8,611	96.3	63,040	31,550	500.8
Iowa	3,886	9,563	4,389	392.9	9,068	4,453	434.9	19,417	10,077	596.9	38,048	18,919	1,424.7
Kansas	7,542	7,885	3,783	1,183.4	6,130	3,074	306.9	12,557	6,263	517.2	26,572	13,120	2,007.5
Kentucky	2,830	22,196	11,473	330.0	3,370	1,659	58.2	16,837	8,244	272.9	42,403	21,376	661.1
Louisiana	6,214	15,663	7,430	105.9	8,505	3,994	65.5	17,727	9,005	201.7	41,895	20,429	373.1
Maine	3,989	6,203	3,420	61.5	817	367	6.0	7,792	4,150	70.2	14,812	7,937	137.7
Maryland	4,557	3,692	1,851	27.4	5,460	3,179	45.8	11,585	5,131	48.2	20,737	10,161	121.4
Massachusetts	2,477	15,424	5,231	34.5	5,845	2,241	1.6	70,124	34,909	59.3	91,393	42,381	95.4
Michigan	6,685	23,665	11,874	571.2	10,898	5,080	260.3	48,853	21,287	363.7	83,416	38,241	1,195.2
Minnesota	4,638	6,596	3,552	957.8	8,705	5,009	659.7	26,327	13,825	735.6	41,628	22,386	2,353.1
Mississippi	8,639	4,194	2,127	152.7	9,555	5,130	227.0	12,772	6,413	374.0	26,521	13,670	753.7
Missouri	10,286	24,661	12,726	752.9	15,498	7,876	401.8	33,337	17,522	553.1	73,496	38,124	1,707.8
Montana	5,163	16,745	10,176	524.2	5,194	3,014	152.2	13,285	7,998	262.6	35,224	21,188	939.0
Nebraska	9,234	12,572	6,640	458.7	5,068	2,480	98.7	16,508	8,598	525.0	34,148	17,718	1,082.4
Nevada	3,631	5,642	4,710	174.1	546	468	34.9	3,141	2,595	130.8	9,329	7,773	339.8
New Hampshire	2,887	2,720	1,534	18.8	406	179	5.7	5,652	2,807	49.9	8,778	4,520	74.4
New Jersey	4,937	12,283	6,131	22.5	9,252	4,612	3.0	15,459	7,303	23.6	36,994	18,046	49.1
New Mexico	3,249	3,673	2,354	122.6	2,520	1,605	46.9	10,181	6,502	215.5	16,374	10,461	385.0
New York	33,162	75,612	39,586	233.9	17,721	8,100	170.9	112,005	52,515	250.2	205,338	100,201	655.0
North Carolina	5,027	16,147	8,010	393.7	5,010	2,379	117.0	28,623	13,924	625.2	49,780	24,313	1,135.9
North Dakota	2,251	11,075	5,657	1,652.8	7,038	3,512	467.8	7,501	3,742	618.4	25,614	12,911	2,739.0
Ohio	14,656	31,484	14,590	276.1	5,333	2,719	163.4	81,954	41,637	342.5	118,771	58,946	782.0
Oklahoma	3,640	16,563	7,620	272.7	5,950	3,306	123.2	23,916	12,088	392.8	46,429	23,014	788.7
Oregon	1,238	3,418	2,060	33.5	4,064	2,230	96.8	16,121	8,981	217.9	23,603	13,271	348.2
Pennsylvania	14,919	17,718	9,124	52.9	15,487	7,739	52.9	86,629	42,704	232.0	119,834	59,567	337.8
Rhode Island	2,056	5,613	2,806	45.7	889	445	2.5	14,673	7,455	17.0	21,175	10,706	65.2
South Carolina	3,345	11,996	6,193	261.5	2,122	1,043	54.5	10,424	5,417	229.9	24,542	12,653	545.9
South Dakota	2,200	8,125	4,754	754.6	5,580	3,056	291.5	10,919	6,603	671.0	24,624	14,413	1,717.1
Tennessee	4,638	12,545	6,147	273.5	8,601	4,033	198.6	24,344	11,540	343.5	45,490	21,720	815.6
Texas	10,453	11,254	5,104	81.7	18,037	9,175	198.2	49,328	23,340	776.2	78,619	37,619	1,056.1
Utah	2,864	4,987	3,764	109.6	1,299	951	39.6	5,478	3,999	165.1	11,764	8,714	314.3
Vermont	1,122	4,636	2,463	61.6	843	427	6.2	4,563	2,242	32.4	10,042	5,132	100.2
Virginia	7,067	19,400	9,551	518.3	11,831	5,905	275.0	14,738	7,236	280.5	45,969	22,692	1,073.8
Washington	3,857	9,927	3,893	174.4	4,384	2,040	97.7	21,323	10,391	132.0	35,634	16,324	404.1
West Virginia	3,819	11,866	6,477	124.1	4,291	2,151	60.1	9,130	4,630	99.8	25,287	13,258	284.0
Wisconsin	7,835	21,029	11,547	555.5	11,178	5,261	203.2	19,129	9,512	482.4	51,336	26,320	1,241.1
Wyoming	1,294	2,044	1,300	48.3	1,976	1,287	60.3	9,363	5,877	276.2	13,383	8,464	384.8
Hawaii	1,452	8,289	3,810	15.3	1,157	378	.5	9,343	3,498	23.7	18,789	7,686	39.5
District of Columbia	3,521	2,864	1,432	.7	2,965	1,482	3.2	1,175	780		7,004	3,694	3.9
Puerto Rico	3,060	10,500	4,902	46.8	4,065	1,961	14.4	8,781	3,910	36.0	23,346	10,773	97.2
TOTAL	316,194	700,314	347,295	14,518.4	334,725	167,670	6,729.3	1,180,775	592,057	14,629.6	2,215,814	1,107,022	35,877.3

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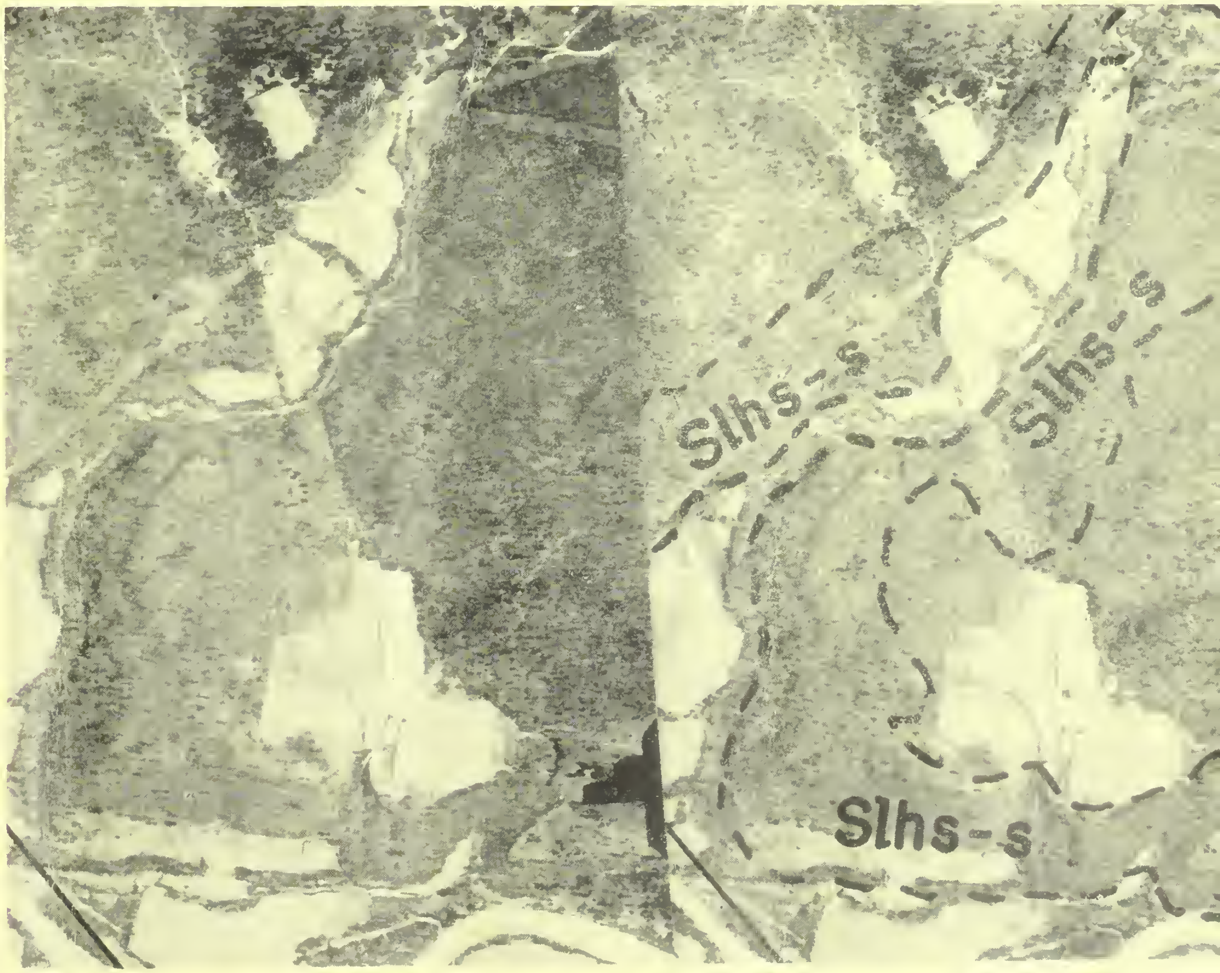
OCTOBER 1951

Public Roads

A JOURNAL OF HIGHWAY RESEARCH



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Drainage and engineering soil maps prepared from aerial photographs aid in the selection of highway locations

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Vol. 26, No. 10 *October 1951*

Published Bimonthly

IN THIS ISSUE

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Inter-American Highway Film 205

Contents of this publication may be reprinted. Mention of source is requested.

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The printing of this publication has been approved by the Director of the Bureau of the Budget January 7, 1949.

BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE
E. A. STROMBERG, Editor

Appraisal of Soil and Terrain Conditions for part of the Natchez Trace Parkway

Reported by PRESTON C. SMITH [1]
Supervising Highway Physical
Research Engineer

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Drainage and engineering soil maps are valuable aids in the selection of highway locations between fixed termini. They are also helpful in the location and design of drainage structures, and provide useful information on soils and other construction materials and on terrain conditions, needed by the engineer in the planning of highway design and construction.

Lacking accurate published maps, aerial photographs provide an excellent base for the preparation of the drainage and soil maps. The proper interpretation of landform, airphoto color tone, drainage characteristics, and other physical features of the aerial photographs, when studied stereoscopically, reveals a great deal about the soil and terrain conditions of the area, particularly when correlated with field investigations and study of available information on the geology and soils of the area.

The study reported in this article, of a section of the Natchez Trace Parkway, was undertaken by the Bureau both to provide information on soil and terrain conditions of the particular area, and also to investigate the technique of airphoto interpretation and to develop a symbol system for the designation of geologic material, terrain conditions, and engineering characteristics of soils.

Introduction

NORMALLY, in highway location, either cultural interests or some outstanding topographic features indicate where a highway must be located at specific points in the region to be traversed. Considerable latitude may be permitted in location of the highway between those points. Standards of design, esthetics, and economy must be considered. Several routes may satisfy the design and esthetic requirements between any two control points, but one route will be more economical than the others. Drainage and engineering soil maps prepared from aerial photographs and other sources of information can be used as aids in the selection of that route. The preparation of strip soil maps for use in determination of the most feasible location of a proposed highway is not new. In 1925, Rose [2] described how such maps could be prepared from soil maps published by the Bureau of Soils, United States Department of Agriculture.

After the location of the highway has been established, the materials engineer must obtain information on soils and other construction materials, and on terrain conditions. The engineering soil map should serve as a guide to determine where field investigations should be concentrated. The collected information will be used in design and construction of the road. The design should be based not only on standardized charts and formulas and the information obtained in the field investigation of this particular area but also on the performance of other roads in areas having similar soil and environmental characteristics.

Although aerial photographs had been used in highway location prior to 1940, they were not used extensively for determination of soil and terrain conditions of highway and airport sites until the advent of World War II. During the last decade several organizations, including the Bureau of Public Roads, Civil Aeronautics Administration, Army Corps of Engineers, Office of Naval Research, and Purdue University, have sponsored projects wherein airphotos were used in identification of soils for engineering purposes. The Bureau of Public Roads has cooperative projects with the State highway departments of New Jersey and Maine for the preparation of soil maps on an area basis, using airphotos insofar as applicable.

In order to determine further the applicability of airphoto interpretation of soil and terrain conditions to highway location and design, the Bureau undertook an engineering soil study of a portion of the Natchez Trace Parkway.

The Natchez Trace, extending from Nashville, Tenn., to Natchez, Miss., was the overland return route, through some 460 miles of forest wilderness and Indian lands, for the crews of flatboats which floated down to New Orleans and were sold there with their cargoes. Its period of use extended from 1798 until 1817, when steamboats began to dominate the Mississippi River traffic.

Congress approved the construction of the Natchez Trace Parkway, extending along the historic route, in 1934. However, the final location of many sections of the parkway has not been made, even though the highway has been completed in some sections and construction is under way on others.

The Bureau anticipated that with the aid of airphotos a generalized engineering report and soil and drainage maps could be prepared which would (1) aid in establishment of the final location of the highway, (2) aid the materials engineers in making a detailed soil survey and locating construction materials along the established highway location, and (3) either aid in determination of a highway design standard for a particular area or, where the design standard has already been established for the area, indicate where the design might need alteration because of local soil or terrain conditions.

There were two secondary objectives. First, it was desired to obtain information concerning some of the techniques involved in the airphoto interpretation of soil and terrain conditions and the preparation of engineering soil strip maps and reports. Second, it was desired to develop a system of symbols for designation of terrain conditions and engineering characteristics of soils. It was believed that the trial area should contain a variety of soil and terrain conditions, and for that reason the proposed 62-mile section of parkway between Gravelly Springs, Ala., and U S 45, near Saltillo, Miss., was selected for study. In that region the final location of the highway had been established only at the termini and at the railroad crossing at Cherokee, Alabama.

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[1] This report is a condensation of a thesis presented to Cornell University in June 1950 in partial fulfillment of the requirements for the degree of Master of Civil Engineering. Captain Smith is at present on military leave, assigned to the Corps of Engineers, Fort Belvoir, Va.

[2] *Field methods used in subgrade surveys,* by A. C. Rose. PUBLIC ROADS, vol. 6, No. 5, July 1925.

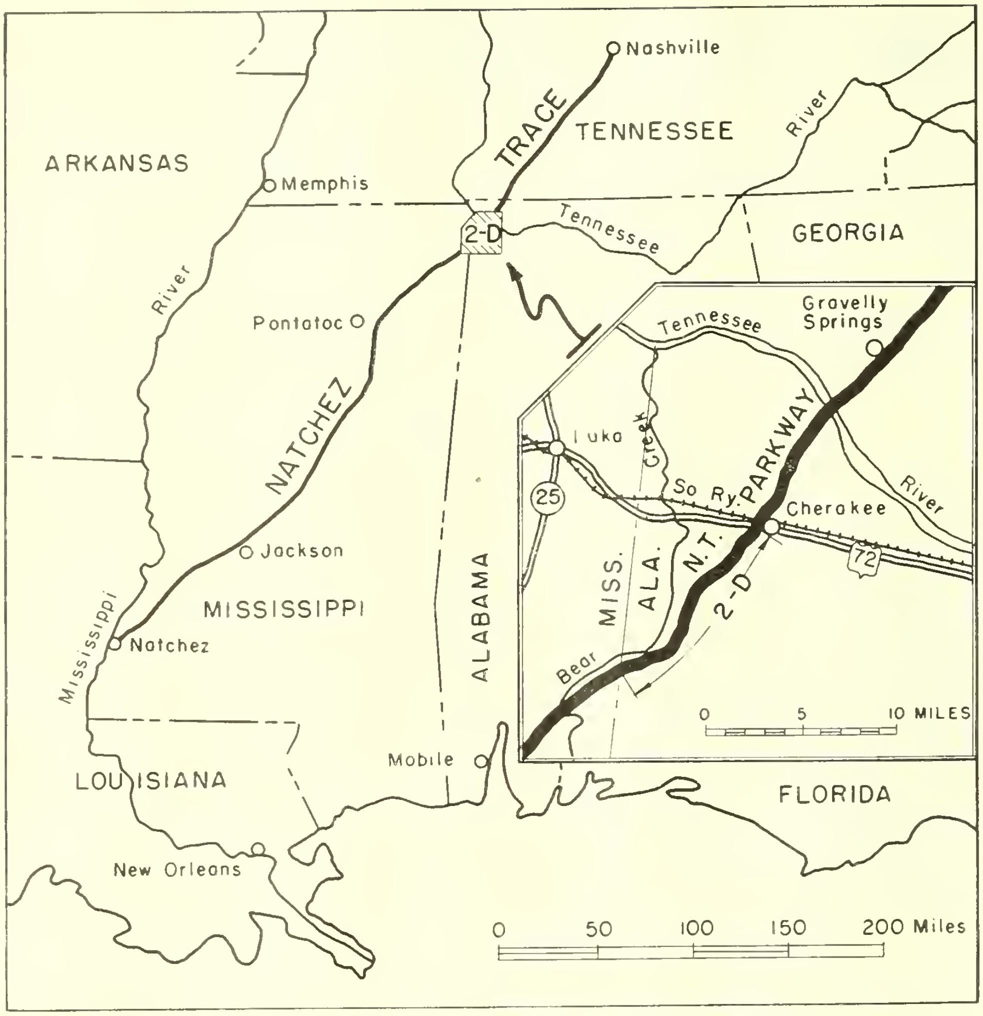

Figure 1.—Location map of the Natchez Trace Parkway and section 2-D.

While this report discusses the principles involved in preparation of drainage and soil maps from aerial photographs, detailed information is confined to section 2-D, one of the six parkway construction sections involved in the basic investigation. As shown in the inset in figure 1, section 2-D extends from U S 72 at Cherokee southwestward to the Alabama-Mississippi boundary line, a distance of approximately 12 miles. The section traverses an area which has considerable variation in topograpy and geology.

Conclusions

Although the accuracy of delineation of the map unit boundaries and description of some of the engineering characteristics of the units produced by this study will not be determined until a more detailed field investigation is made and actual construction is undertaken, it is believed that the following general statements regarding the use of the maps prepared primarily from aerial photographic information are warranted:

1. Where there are two or more possible alternate routes for the highway, soil and drainage maps will aid in determination of the best route.
2. The engineering soil map will aid the materials engineer in locating places at which an extensive soil survey should be made, probable sources of borrow, and possible sources of aggregate.
3. When the engineer has limited knowledge concerning construction materials in a region, the soil map and description of soil map units will provide information useful in the design of the highway section and pavement.
4. The drainage map can be used to determine the locations of and the areas to be served by highway drainage structures.
5. The soil map is generalized and, as a consequence, small, isolated areas of map units have not been delineated. A reasonably precise boundary sometimes cannot be established in the transition zone between some map units, particularly when material from a higher unit is washed onto a lower unit.
6. When the ground water condition or internal drainage of a unit is of particular importance to the highway engineer, he should make a field investigation of the condition. Determination of the normal internal drainage potential of a unit is difficult to determine from airphotos, particularly when there has been considerable rainfall just prior to the time the photographs were taken.

The following conclusions were reached in regard to techniques involved in analysis of soil characteristics and terrain conditions and preparation of soil and drainage maps:

1. The system of symbols proposed by Lueder and used with some modification in this report satisfactorily portrays the most important soil profile and environmental characteristics of the map units. The system of symbols can readily be mastered, for the symbols are suggestive of particular characteristics of map units.
2. By airphoto interpretation alone, it is sometimes impossible to determine in which of two similar soil groups the map unit soil should be placed. For example, differentiation cannot usually be made between A-2 and A-3 or A-6 and A-7 soils without field investigation.
3. While it is sometimes necessary to use more than one soil group symbol to indicate the variation in the soil profile, the use of the multiple numbers is an excellent indication of extreme variations in the soil strata.
4. The preparation of soil and drainage maps at airphoto scale by making an airphoto overlay, followed by photographic or some other method of reduction to a desired scale, is preferred when a base map must also be prepared. When a base map is available, however, some other method of transfer of the airphoto-delineated soil boundaries and drainage lines to the base map may be preferred.

Drainage Map

A drainage map shows the location of streams and their drainage areas, thus aiding the engineer in the determination of the proper location and design for drainage structures. Drainage channels can readily be discovered by stereoscopic examination of airphotos. In special cases, the map aids in determining the extent and characteristics of soils or geologic materials.

For the benefit of the highway engineer, streams and soil boundaries shown on drainage and soil maps should be located with respect to either adjacent cultural features or established ground control points. No large-scale maps suitable for these purposes have been published which cover all of the area under consideration. A base map was therefore prepared from airphotos, having a scale of approximately 3 inches to 1 mile, using contact prints obtained from the Production and Marketing Administration of the U. S. Department of Agriculture. These provided stereoscopic coverage for a 3-mile width of land containing the preliminary line of the parkway.

Normally, the successive airphotos in a flight strip overlap more than 50 percent, while adjacent flight strips overlap about 20 percent. In making the overlay, since the greatest distortion (perspective displacement) of features occurs at the margins of photographs, only the central portion of each photograph was used. The base map has slight variations in scale

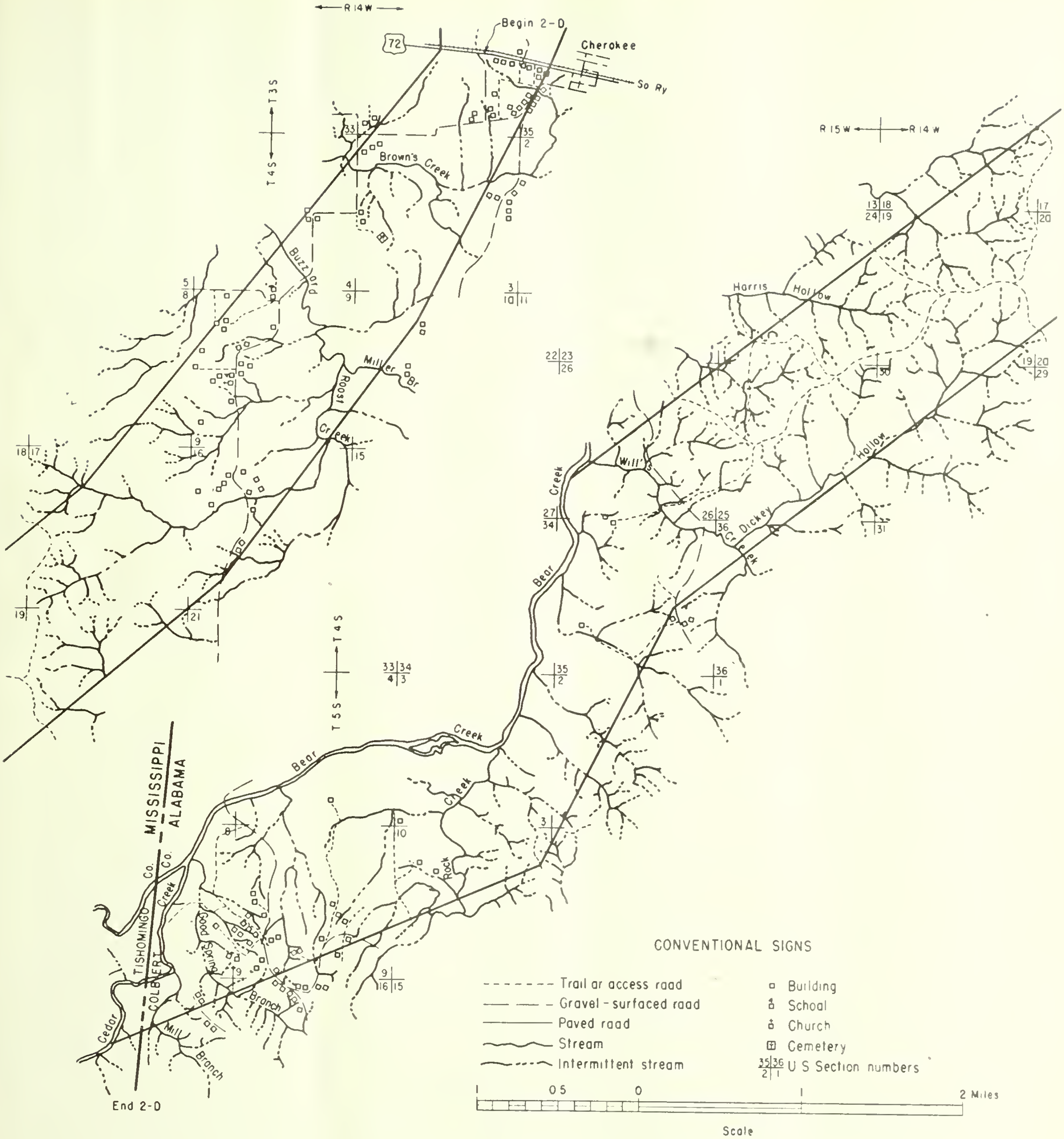

Figure 2.—Drainage map of section 2-D.

because distortion increases with distance from the center of the airphoto and the scale at the center of one photograph may not be exactly the same as that of another photograph. However, if an average scale is used and measurements on the maps are made from the nearest cultural feature, points or boundaries on the maps can be located with reasonable accuracy in the field. With a limited amount of field work and identification of cultural features on airphotos, photogrammetric methods could have been used to produce a more accurate base map. However, it was decided that the greater expenditure of effort would not be justified by the limited additional benefit derived.

The preliminary survey line (P-line) having been marked on the airphotos, approximately parallel boundary lines for mapping were established by connecting arbitrarily chosen points on each side of, and about one-half mile from the P-line. Although the final location of the highway may vary from the P-line in some areas, it will usually be contained within the mapped strip. Hereafter, in this report, when reference is made to section 2-D, the section is considered to extend across the entire width of the 1-mile strip, even though the section number was originally assigned only for the parkway width.

A sheet of transparent cellulose acetate was placed on an airphoto and a tracing made of cultural features such as roads, houses, and cemeteries within the 1-mile strip. Adjacent prominent features were also traced. Most of the roads could be traced by use of the base photograph, but to locate some of the buildings it was necessary to use the adjacent photograph (in line of flight) and view the pair stereoscopically. Using United States Geological Survey topographic maps insofar as possible, and soil survey maps of the United States Department of Agriculture or State Highway Planning Survey maps for the remainder of the area, sufficient features on the maps were identified on the airphotos to permit the location of most of the United States public land section lines on the acetate sheet. Some of the field lines on the airphotos coincided with section lines, but "bridging" was necessary in uncultivated areas.

With the acetate sheet still secured to the base photographs, the 1-mile strip was viewed stereoscopically (using pairs of photographs) and the drainage lines traced. A solid or continuous line was used for a permanent stream, while a broken line was used for intermittent drainage. The distinction between permanent and intermittent drainage was based on drainage area, length of stream, channel size, character of soil, and geology.

When the tracings from individual photographs were completed for the entire construction section, they were assembled into a strip map and oriented north-south and east-west in conformity with the United States section lines. A finished map was then made from the assembled individual tracings. Figure 2 shows the drainage map for section 2-D.

On a strip drainage map, a watershed can usually be determined only for the small streams, since watersheds for the large streams extend beyond the boundary of the mapped area. However, if the design engineer is interested in a particular watershed which is not completely shown on the strip map, the complete drainage system can be studied stereoscopically on the airphotos.

Buzzard Roost Creek (near the north end of section 2-D) and downstream sections of its major tributaries are on soluble limestone; hence, there is but little surface drainage. However, dendritic drainage patterns have developed near the source of the major southern tributaries of the creek where the soils have developed from either unconsolidated coastal plain material or calcareous shale.

Except for areas along Bear Creek (near the south end of the section) and near the mouths of its major tributaries, the dendritic pattern is continued through the southwestern two-thirds of section 2-D. Surface drainage is not well developed on alluvial areas. Also, only a few tributary streams have developed where bedrock is exposed in the lower portions of valley walls.

Engineering Soil Map

Knowledge of the physical properties of soils alone is not sufficient for the solution of many engineering problems involving soils. Hence, in order to be of maximum value to the highway engineer, a soil map is needed that not only shows the character of the soil but also indicates its environmental characteristics. Such information can usually be obtained by stereoscopic study of airphotos and interpretation of airphoto patterns.

A number of authors[3] have described the significant factors involved in interpretation of airphoto patterns which reflect soil characteristics and terrain conditions, and the details are therefore not presented here. In accordance with the procedure usually recommended, the interpretation of airphoto patterns on the Natchez Trace section started with the fundamental concept that, when subjected to the same climatic conditions, similar landforms have similar airphoto patterns. The airphoto patterns were analyzed in order to subdivide the landforms into units which would be significant to the highway engineer. Drainage, erosion, topography, airphoto color tone, vegetation, and cultural features were considered.

The landform of a geologic formation is influenced by climatic and biotic factors, relief, time, and inherent properties of the material. The climate is relatively uniform in the area under investigation; hence, little difference in weathering of similar materials having similar topography should occur. The mean annual rainfall at Tuscumbia, Colbert County, Ala., is 49.4 inches. October is the driest month, with an average rainfall of 2.8 inches, while March, the wettest month, has 5.8 inches. The mean annual temperature at Tuscumbia is 61° F., and the mean temperature for the coldest month, January, is 42° F.

Interplay of the other factors in varying amounts on each parent formation has resulted in various landforms within the mapped area.

Symbols Characterize Map Units

The term "map unit" is used to describe an area in which landform, parent material, soil profile, topography, and drainage are relatively uniform. Soil mapping, then, involves obtaining information concerning those elements and delineating boundaries of areas in which they are uniform.

The map symbols used should readily convey the desired information to the engineer. Lueder[4] has presented a system of map-unit symbols developed during preparation of engineering soil maps of areas in New Jersey. His basic system was applied in the Natchez Trace mapping, with some modifications; the principal modification being the use of a dash to separate factors pertaining to landform and geologic material from factors pertaining to soil profile and topographic conditions. A symbol was also introduced to describe ground slope, which is of value for location, excavation, and run-off determinations. The symbols used to describe map-unit characteristics in Natchez Trace mapping are as follows:

Parent formation:

- *S* . . . Sedimentary (consolidated)
    - *h* . . . Shale
    - *l* . . . Limestone
    - *s* . . . Sandstone
- *A* . . . Alluvial
    - *T* . . . Terrace
    - *R* . . . Recent
    - *O* . . . Old
- *W* . . . Windblown
- *M* . . . Marine (coastal plain)
- *V* . . . Variation (in thickness, density, or composition of strata; used only on the left side of the dash).

Internal drainage:

- *e* . . . Excellent (granular material; ground water table at such depth that it is not significant).
- *g* . . . Good (permits traffic or excavation soon after rain; position of ground water table rarely significant).
- *i* . . . Imperfect (trafficability usually poor and excavation impractical during significant periods; has occasional high ground water table, particularly in alluvium; raised grade line usually desirable in lowlands; artificial drainage and placement of either subbase or heavy base course may be required for flexible pavements in cut sections).
- *p* . . . Poor (ground water table usually near ground surface; raised grade line necessary in lowlands; artificial drainage may be required and placement of either subbase or heavy base course is usually required for flexible pavements in cut sections; excavation difficult during the winter season and for a considerable period after a rain).

Ground slope:

- *l* . . . Level (range in slope from 0 to 3 percent).
- *u* . . . Undulating (usually between 3 and 10 percent but may have some

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[3] *The engineering significance of soil patterns*, by D. J. Belcher. Proceedings of the Highway Research Board, vol. 23, 1943, p. 569. *The origin, distribution, and airphoto identification of United States soils*, by D. J. Belcher, D. S. Jenkins, L. E. Gregg, and K. B. Woods. Technical Development Report No. 52, Civil Aeronautics Administration, 1946. *The engineering significance of landforms*, by D. J. Belcher, Bulletin No. 13, Highway Research Board, 1947, p. 9. *Aerial photographs: their use and interpretation*, by A. J. Eardley. Harper & Brothers, 1942. *Identification of granular deposits by aerial photography*, by R. E. Frost. Proceedings of the Highway Research Board, vol. 25, 1945, p. 116. *Aerial photographs and their applications*, by H. T. U. Smith. D. Appleton-Century Company, Inc., 1943.

[4] *A system for designating map-units on engineering soil-maps*, by D. R. Lueder. Bulletin No. 28, Highway Research Board, 1950, p. 17.

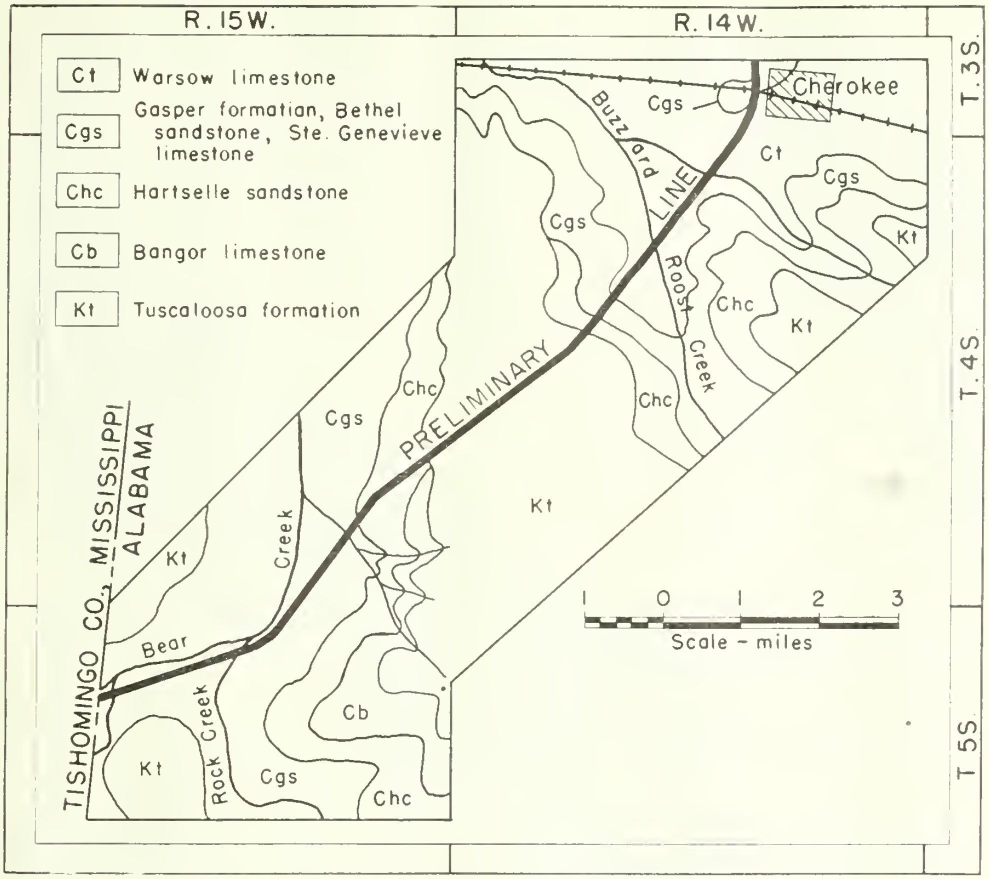

Figure 3.—Geologic map of area traversed by section 2-D.

flatter or steeper slopes; steeper slopes are short).

s . . . Steep (most of slopes steeper than 10 percent, but may have small areas with flatter slopes).

Highway Research Board soil classification.[5] Group number is indicated by a numeral, 1 to 7. Only predominant groups, referring to material within significant depth, are shown in map unit designation. Highway Research Board subgroup numbers, such as 7–6, are not used. Combined numerals, such as 24, indicate that the soils of the map unit vary, either laterally or vertically, but are principally in A-2 and A-4 groups.

Horizon: The Symbol *C* indicates significant difference between B and C horizons and is placed on the right side of the dash since it is a soil profile symbol.

When the mapping is so generalized that one symbol will not adequately describe a particular characteristic of the map unit, a combination of symbols may be used. For example, *us* on the right side of the dash indicates that ground slopes of the map unit vary from undulating to steep. The combination *ls* on the left side of the dash indicates alternate beds of limestone and sandstone, neither of which has been separated into an individual map unit. If the limestone overlies the sandstone, the combination should be *l/s*.

The internal drainage symbol indicates the drainage potential of the soil due, primarily, to its textural and structural properties. However, in landforms for which the ground water table is a factor in highway location, design, or construction, the symbol also reflects the approximate depth to the water table during prolonged wet periods. By reference to the other symbols of the map unit, it is usually possible to determine if a particular internal drainage symbol denotes a drainage condition which is due primarily to soil properties. By use of airphotos, without field investigation, it is impossible to estimate the precise depth to the ground water table for most of the map units, since an area may have been photographed after a considerable dry period when the water table is low.

An important advantage of the developed system is that the symbols are applicable to certain engineering factors which may be ascertained from geologic reports, topographic maps, and agricultural soil bulletins. Thus, published material can be used to supplement the information obtained from aerial photographs. The type of parent material and some idea concerning the

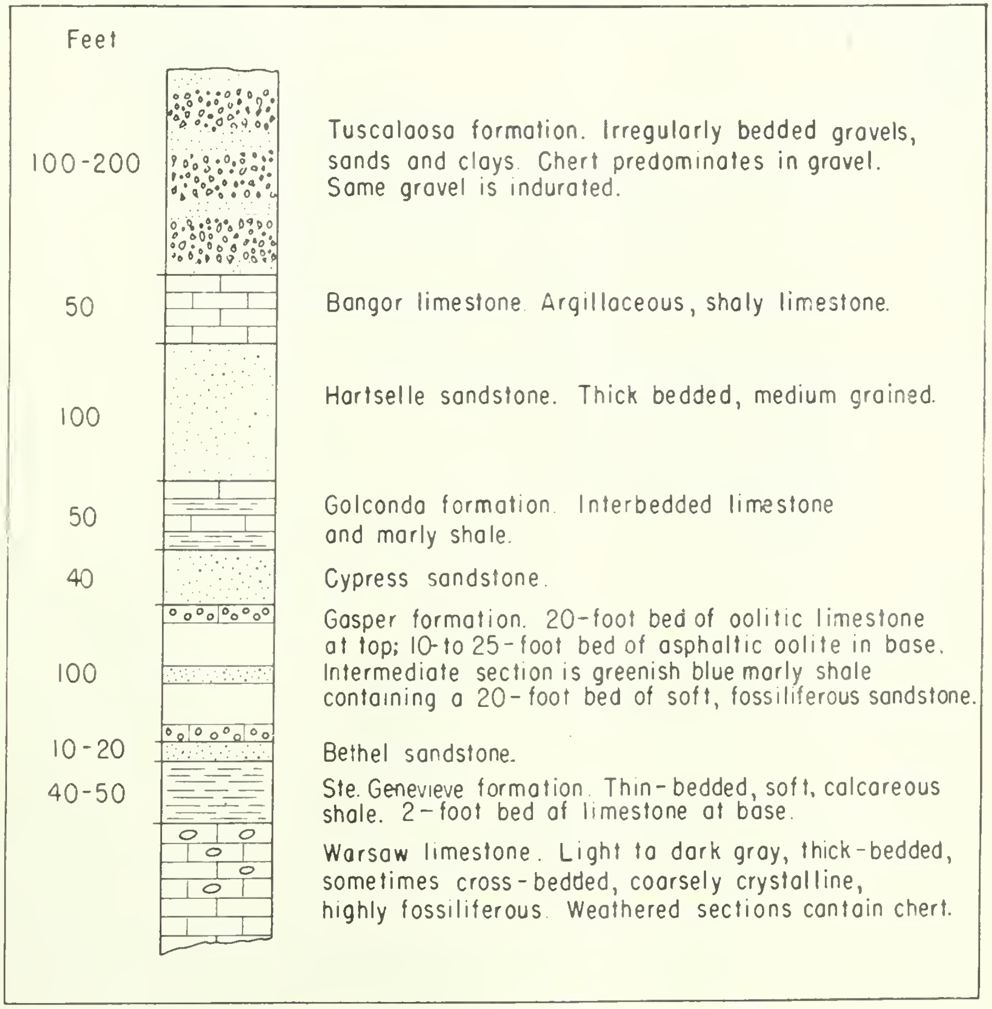
Feet
100-200
Tuscaloosa formation. Irregularly bedded gravels, sands and clays. Chert predominates in gravel. Some gravel is indurated.
50
Bangor limestone. Argillaceous, shaly limestone.
100
Hartselle sandstone. Thick bedded, medium grained.
50
Golconda formation. Interbedded limestone and marly shale.
40
Cypress sandstone.
100
Gasper formation. 20-foot bed of oolitic limestone at top; 10- to 25-foot bed of asphaltic oolite in base. Intermediate section is greenish blue marly shale containing a 20-foot bed of soft, fossiliferous sandstone.
10-20
Bethel sandstone.
40-50
Ste. Genevieve formation. Thin-bedded, soft, calcareous shale. 2-foot bed of limestone at base.
Warsaw limestone. Light to dark gray, thick-bedded, sometimes cross-bedded, coarsely crystalline, highly fossiliferous. Weathered sections contain chert.

Figure 4 (at right).—Approximate geologic columnar section through highlands southwest of Cherokee, Ala.

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[5] *Classification of highway subgrade materials*, Report of Committee on Classification of Materials for Subgrades and Granular Type Roads, Proceedings of the Highway Research Board, vol. 25, 1945, p. 375.

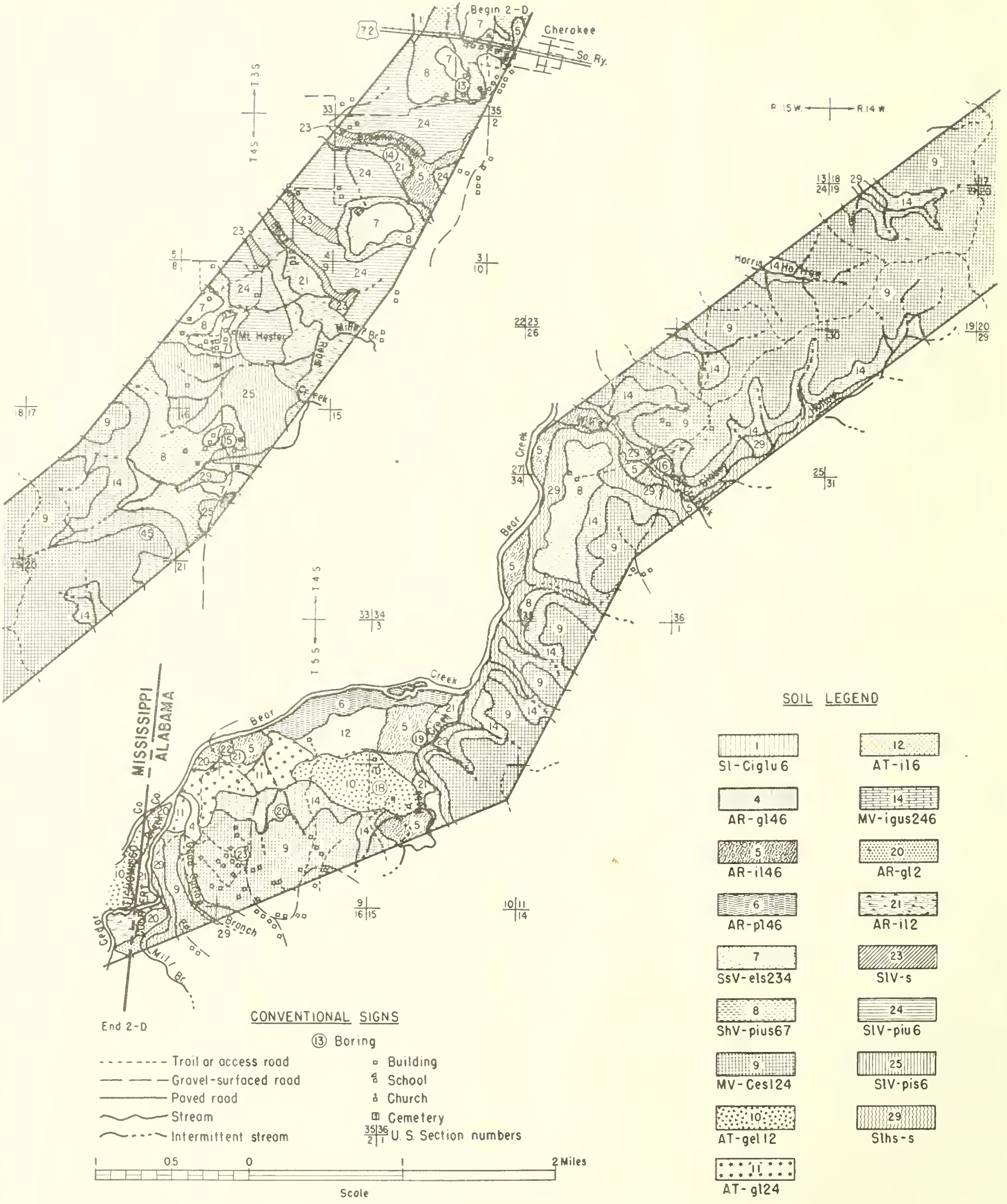

Figure 5.—Engineering soil map of section 2-D.

derived soils can be determined from geologic reports and maps. Ground slopes and major drainage channels are shown on topographic maps. The agricultural soil bulletins give a description of soil profile, topography, internal drainage, and generalized geology. Some of the engineering characteristics indicated by other basic sources of information may be precisely determined by the use of aerial photographs. The remainder of the factors can be inferred by study of airphoto patterns. Use of geologic and agricultural reference material reduces the amount of field work necessary to verify the inferences.

Mapping Procedure

The initial office study consisted of two parts. First, geologic and soil[6] reports concerning the area to be traversed by the Parkway were studied. Second, a preliminary stereoscopic study of airphotos for the area was made, including a generalized correlation of airphoto patterns with various parent materials, soils, and terrain conditions likely to be encountered in parkway construction. Road cuts and other exposures in the vicinity of the proposed parkway where a soil profile for a typical airphoto pattern or parent material could be observed were marked on the airphotos.

Following the office study, the areas marked on the airphotos were inspected in the field. If the soil profile or ground condition was that anticipated from airphoto study, or was in conformity with that exhibited for a similar airphoto pattern already investigated in another area, no further study of that area was necessary. However, if a soil profile or condition differed from that anticipated from the airphoto pattern, further study was undertaken in order to determine the features of the airphoto pattern which would depict the soil profile and ground condition. The field reconnaissance resulted in correlation of airphoto patterns with profile and other ground conditions to such an extent that a tentative definition of soil map units for the entire section could be made.

Using a red China-marking pencil, boundaries of soil map units were marked on the same photographs from which the base map was prepared. Each soil area was indicated by a number which corresponded to a definite map unit. An area in which there was some question about either the map unit or its boundaries was marked on the adjacent photograph, in order that it could be given further office or field study. By additional study of geologic and soil reports, the map unit boundaries were established for some of the questionable areas.

Each of the remaining unmapped areas was studied in the field. Soil profiles were examined in road cuts where possible. Otherwise, borings were made. Boundaries were either marked on the airphotos at that time or sufficient information was obtained to permit office delineation.

Typical map unit locations were also examined in the field. For map units having appreciable depth of soil, a sample was usually taken from each significant soil stratum below the A-horizon. Samples were also taken to show the variations in physical properties of soils within the map unit. If borings or field observations showed that map unit boundaries or descriptions were inaccurate, corrections were made on the airphotos in the field or in the office.

A final definition of map units was made after the soils had been tested in the laboratory. The soil map was then prepared as described below. Work was done on the detailed report concurrently with soil testing and map drafting.

Geology of the Area

The topography of the area traversed by section 2-D is quite variable. The ground surface ranges from relatively narrow ridges with steep slopes, some elevations in excess of 700 feet, and local relief of as much as 300 feet, to low rounded hills separated by relatively broad valleys. Consequently, a variety of geologic formations of Mississippian and Upper Cretaceous age are exposed in the area.[7] Figure 3 is a geologic map of section 2-D, and a columnar section through the formations is shown in figure 4.

Preparation of Soil Map

After field investigation and laboratory testing of soils verified that the airphoto-delineated map units were correct, a soil map was prepared. A tracing was made of cultural features and principal streams from the drainage map already completed. Locations of borings from which soil samples were taken were also shown on the tracing. The marked airphotos were then properly positioned beneath the tracing and the soil map unit boundaries traced. The completed map for section 2-D is shown in figure 5.

On the soil map a number and a conventional map pattern are used to identify each delineated map unit. The numbers have no inherent significance. They merely serve as a quick means for identifying the map patterns, as will be seen from the soil legend in figure 5.

The scale of the map depends upon the use to be made of it and the complexity of the mapped area. If map units are large and only a few units are encountered, relatively small-scale maps—even less than an inch equals 1 mile or smaller—may suffice. Maps having a scale of at least 2 inches to 1 mile are usually preferred by the field engineer in order that he may record reconnaissance notes on the map. However, since the detail on the map cannot be greater than that of the airphotos, the maximum practical scale is that of the latter (approximately 3 inches to 1 mile for this study).

Characteristics of Map Units

It is understood, of course, that on a soil map prepared primarily by airphoto interpretation the information is generalized. While the map and description of map units for section 2-D may be sufficiently detailed for use in preliminary location and estimates, some portions of the final location will require more detailed information. The soil map will usually indicate where the field investigation of soil and ground conditions must be concentrated.

The engineering soil map units delineated in section 2-D differ in some characteristics which are of importance to the highway engineer. Even though some of the landforms were divided into several maps units, it was revealed by borings, field observation of soil profiles in road cuts, and laboratory soil test results that considerable variation both laterally and vertically exists within some units. Some of the variations are described in the discussion of the map units which follows. More detailed information concerning the variations in soil profiles within the map units may be obtained by reference to the summary of soil test results shown in table 1. Since the original mapping was for a 62-mile portion of the parkway and soil samples were taken which would be representative of map units within that length, some map-unit soils were not sampled in section 2-D. Hence, table 1 shows some soil test results for map units occurring in section 2-D but for which the representative soil samples were taken from another construction section.

The probable Highway Research Board group number was used in the description of the soil for those map units having appreciable soil mantle. This was considered permissible in such highly generalized mapping, even though the stated soil groups were based on a limited number of laboratory tests and field observations. The test data given in table 1 support the estimates.

The map unit descriptions do not indicate whether materials may be suitable for use as aggregates in bituminous mixtures or in portland cement concrete. The determination of quality, availability, and extent of aggregates must be made in a detailed survey by the materials engineer.

Although some of the engineering characteristics of map units may be ascertained by direct stereoscopic observation of airphotos, other characteristics are deduced from features of the airphoto pattern. Figures 6–14 show the airphoto patterns of the most extensive map units in section 2-D.

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[6] *Soil survey, Colbert County, Alabama.* U. S. Department of Agriculture, Bureau of Chemistry and Soils, Series 1933, Number 22, issued February 1939.

[7] Information concerning geology of the area traversed by Section 2-D was obtained from *Geology of Alabama*, by G. I. Adams, C. Butts, L. W. Stephenson, and W. Cook. Special Report No. 14, Geological Survey of Alabama, 1926. Figures 3 and 4 are based on this report.

Table 1.—*Representative soil test data for section 2-D map units*

Map unit designation	Boring number[1]	Depth of sample below surface	Soil horizon	Mechanical analysis: cumulative percentage passing—							Physical test constants				Moisture-density		Highway Research Board classification	
				¾-inch	⅜-inch	No. 4	No. 10	No. 40	No. 200	Smaller than 0.005 mm. (clay)	Liquid limit	Plasticity index	Shrinkage limit	Shrinkage ratio	Maximum dry density	Optimum moisture	Group index	Group
		Inches													*Lb./cu. ft.*	*Percent*		
Sl-Ciglu6	2	48–72	C	100	99	99	98	96	83	37	38	15	16	1.9	107	18	10	A–6
Do	8	15–28	B	----	----	----	100	99	78	39	40	16	15	1.9	110	17	10	A–6
SsV-els 234	11	18–24	B	----	----	----	100	99	71	29	28	11	15	1.9	----	----	7	A–6
Do	11	42–60	C	----	----	----	100	99	69	35	34	14	13	1.9	----	----	8	A–6
Do	11	72–90	D	----	----	----	100	99	59	36	35	13	14	1.9	115	15	6	A–6
Do	11	96–108	E	----	----	----	----	100	37	23	25	8	19	1.8	----	----	0	A–4
Do	11	114–132	F	----	----	----	----	100	32	13	19	NP	18	1.8	117	12	0	A–2–4
ShV-pius 67	13	28–36	B	100	95	93	91	89	49	32	33	15	16	1.9	114	14	4	A–6
Do	13	56–66	C	----	----	----	100	99	77	37	38	16	15	1.9	111	18	10	A–6
Do	13	84–96	D	----	----	----	100	99	73	29	31	11	16	1.8	----	----	8	A–6
Do	15	8–15	B	----	----	----	100	99	95	81	77	47	14	1.9	----	----	20	A–7–5
Do	15	20–28	C	----	----	----	100	99	90	64	64	36.	12	1.9	----	----	20	A–7–6
MV-Ces 124	23	10–30	B	99	98	97	96	91	81	35	36	15	17	1.8	116	16	10	A–6
Do	23	36–60	C	85	55	37	30	25	11	28	32	11	25	1.6	112	16	0	A–2–6
Do	45	10–30	B	96	87	73	63	54	45	32	42	21	16	1.8	----	----	5	A–7–6
Do	45	36–60	C	100	98	96	95	71	26	22	29	11	18	1.8	----	----	0	A–2–6
Do	45	96–120	D	87	71	54	43	18	15	31	47	23	21	1.7	122	12	0	A–2–7
MV-igus 246	20	10–18	B	----	----	----	100	98	85	34	35	13	17	1.8	----	----	9	A–6
Do	20	24–36	C	----	100	99	97	93	75	28	29	10	16	1.8	120	14	8	A–4
Do	20	48–60	D	92	81	73	66	58	42	30	32	13	16	1.9	115	14	2	A–6
Do	20	80–96	E	99	96	93	89	85	72	45	51	30	13	1.9	111	18	17	A–7–6
Do	20	108–120	F	----	----	----	100	97	89	56	50	30	11	2.0	----	----	18	A–7–6
AT-gel 12	18	10–18	B	96	93	90	86	76	55	30	34	16	15	1.9	----	----	6	A–6
Do	18	20–36	C	99	85	73	63	47	31	33	42	19	17	1.8	----	----	1	A–2–7
AT-gel 24	21	8–15	B	----	----	----	100	98	64	29	32	13	13	1.9	----	----	7	A–6
Do	21	36–48	C	----	----	----	100	98	48	26	27	10	15	1.9	122	13	3	A–4
Do	21	84–108	D	----	----	----	100	99	45	23	24	8	17	1.8	122	12	2	A–4
AT-il 6	34	12–18	B	----	----	----	100	99	80	32	38	17	18	1.8	----	----	11	A–6
Do	34	36–48	C	----	----	----	100	99	67	27	31	13	15	1.9	----	----	7	A–6
AR-gl 2	22	12–18	B	----	----	----	100	84	16	6	NP	NP	----	----	----	----	0	A–2–4
Do	22	36–48	C	----	----	----	100	84	15	5	NP	NP	----	----	111	12	0	A–2–4
AR-il 2	14	24–30	B	----	----	----	----	100	63	35	34	16	14	1.9	----	----	8	A–6
Do	14	48–54	C	----	----	----	----	100	34	19	22	7	16	1.9	----	----	0	A–2–4
AR-il 46	19	8–18	B	----	----	----	----	100	95	46	42	19	19	1.8	----	----	12	A–7–6
Do	19	48–72	C	----	----	----	----	100	80	29	28	10	18	1.8	118	14	8	A–4

[1] Approximate locations of borings are shown on figure 5. Borings Nos. 2, 8, 11, and 34 were made in other sections of the parkway.

In the following descriptions of the map units encountered in section 2-D, some units are grouped on the basis of geologic material while others are grouped according to landform.

Units on cherty limestone

Sl–Ciglu 6.—The unit has level to undulating topography underlain primarily by slightly friable A-6 soil to a depth greater than 15 feet and containing angular chert, which ranges up to 8 inches in size and to 25 percent of total material, in the subsoil. Soil from the B-horizon, which extends to a depth of 3 to 8 feet, is more plastic than from the C-horizon and soils from both horizons may be A-7. Much of the surface water is collected in depressions or sinkholes and removed through subterranean channels. In undisturbed condition the soil has good internal drainage; remolding destroys the soil structure and decreases the permeability. Bedrock occurs at such great depth that it is not a source of aggregate. Airphotos show the mottled sinkhole pattern and general lack of surface drainage development. The unit is usually cultivated.

SlV–piu 6.—The airphoto pattern of this unit is illustrated in figure 6. The topography is generally undulating, with some short steep slopes, some limestone depressions, and underlain by bouldery, plastic A-6 soil; bedrock normally occurs at a depth of less than 10 feet. Limestone boulders on or near the ground surface will interfere with earth work. Depressed areas have a high ground-water table during substantial periods. Much of the surface water is collected in depressions and discharged through subterranean channels; hence, some faintly discernible gullies disappear at the edge of depressions. Although limestone might be quarried from this unit and used for road surfacing, the quarry opening will normally be made in the adjacent unit, *SlV–s*. The airphoto color tone is medium gray mottled with darker gray. On nearly level areas the dark areas are connected to form a phantom drainage pattern.

SlV–pis 6.—The airphoto pattern of this unit is also shown in figure 6. The topography is rolling and underlain by bouldery, plastic A-6 soil, with limestone bedrock normally at a depth of less than 10 feet. Limestone boulders, exposed in some areas, will interfere with earthwork. Although the internal drainage is poor to imperfect, surface drainage is good. Extensive surface drainage channels have been formed because of the steep ground slopes.

SlV–s.—The unit is composed of valley walls in which limestone is either exposed or near the ground surface; slopes vary from 10 percent in shallow soil areas to nearly vertical in rock exposures. Where there is a soil mantle it is shallow and contains angular chert or cherty limestone fragments. Bedrock exposures may be sources of aggregate. On airphotos the identity of

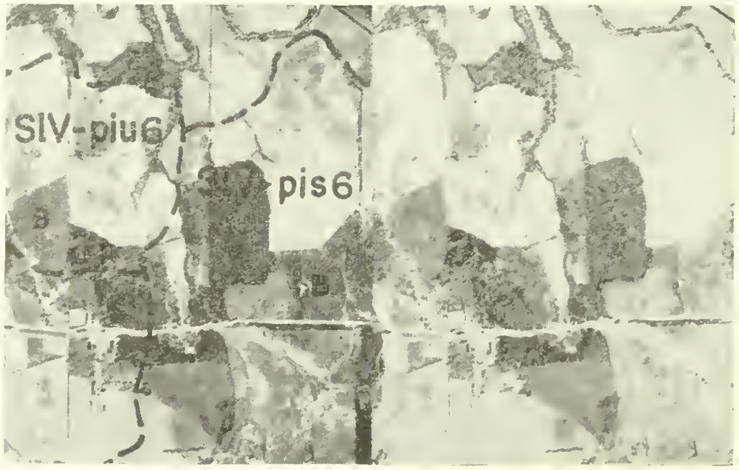

Figure 6.—Airphoto stereo-pair illustrating map units **SlV-piu 6** *and* **SlV-pis 6.** *Boulders are exposed in some areas as at* **B.**

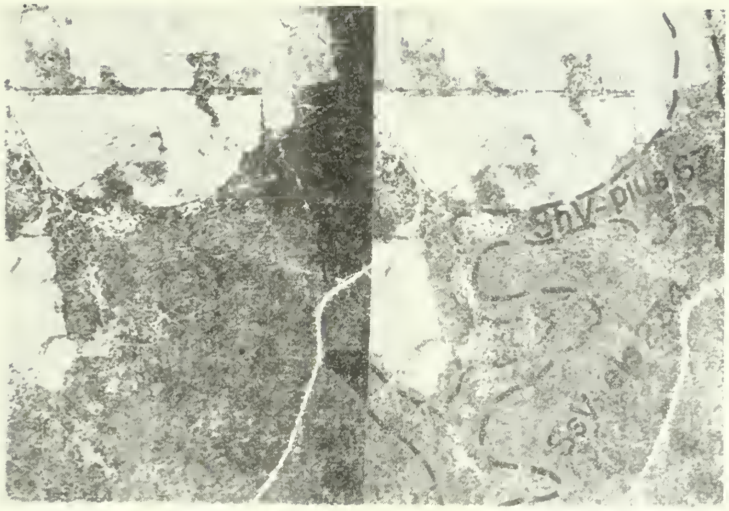

Figure 7.—Airphoto illustrating map units SsV-els 234 *and* ShV-pius 67.

this unit is established by determination that adjacent units are derived from limestone.

Units on sandstone, calcareous shale, and interbedded sandstone, shale, and limestone.

SsV–els 234.—The airphoto pattern of this unit is illustrated in figure 7. The unit occupies tops of hills and ridges, usually wooded; ridge tops may be more than 100 yards wide and are nearly flat, but side slopes are steep. Normally, weakly cemented, weathered sandstone occurs at a depth of 2 to 4 feet, the C-horizon soil is A-2 or A-3, and the B-horizon soil is friable A-4. However, some hills and ridges of this unit are capped with coastal plain soils, and weathered bedrock may not be encountered within a depth of 10 feet. The A-2 or A-3 soil and weathered sandstone may be used in a subbase but the sandstone is usually not of such quality that it can be used for base course or surfacing material. However, if an exposure of asphaltic sandstone is located, its suitability as aggregate should be investigated. Neither internal nor external drainage should be a problem in this unit. Watersheds are small and most of the rainfall is absorbed by the porous soil; hence drainage channels are not well developed. A landscape view is shown in figure 8.

ShV–pius 67.—The airphoto pattern of this unit is also shown in figure 7. The unit is either woodland or pasture. It occupies 5- to 20-percent slopes and normally has a mantle of elastic A-6 or A-7 soil, 2 to 10 feet in depth, with poor or imperfect internal drainage. Erosion has exposed the weathered shale in some areas. Within the usual depth of cuts the weathered shale is easily excavated; blasting will be required in deep cuts. Some areas of this unit are underlain by interbedded shale and limestone and excavation in the limestone will require blasting. The limestone, particularly that containing asphalt, may be suitable base course or surfacing material.

Slhs-s.—The airphoto pattern of this unit is shown in figure 9. The unit has slopes which usually range from 20 to 90 percent and are underlain by undifferentiated limestone, shale, or sandstone formations. The materials are exposed as nearly vertical cliffs in valley walls or occur on such steep slopes that bedrock will be encountered in shallow cuts. The flatter slopes probably have some soil mantle as well as a variable depth of weathered rock; delineation of each material cannot be made on the airphoto. Well-defined stream channels have been cut in the rock slopes, while the small tributary streams at higher elevation have no well-defined channels where there are porous, coastal-plain soils. Unweathered sandstone or limestone, particularly asphaltic limestone, may be suitable base course or surfacing material. Most of the areas are wooded.

Units on coastal plain deposits

MV-Ces 124. — The airphoto pattern of this unit is shown in figure 10. The unit occupies dissected upland areas underlain by stratified A-1, A-2, and occasional A-4 or even A-6 soil. Chert gravel predominates and usually occurs in strata having a thickness of 5 to 20 feet or more, with maximum particle size of about 4 inches. The unit contains the most extensive deposits of possible base course gravel in the region, although excessive clay, induration of gravel, and poor gradation precludes the use of many exposed deposits. Both surface and internal drainage of the unit are usually good but seepage may occur at the top of plastic soil strata. The ridge tops have undulating topography but the side slopes are steep and relatively straight. Local roads and trails on the ridges are quite circuitous. Gullies and minor tributary streams pursue relatively straight courses.

MV-igus 246.—The airphoto pattern of this unit is also shown in figure 10. The unit occurs downslope from *MV-Ces 124* and consists of material washed from the

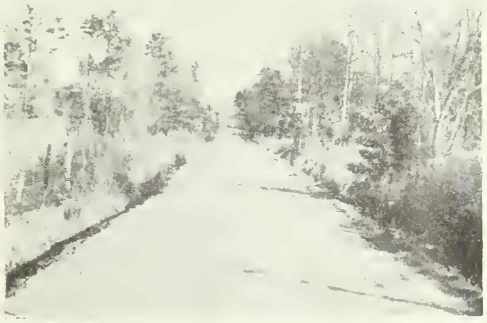

Figure 8. — Easily weathered calcareous shales are usually exposed below a more resistant formation. Here, the steep area in the distance is map unit SsV-els 234 *(soil formed from sandstone) while the flatter slope in the foreground has developed on the shale unit* ShV-pius 67.

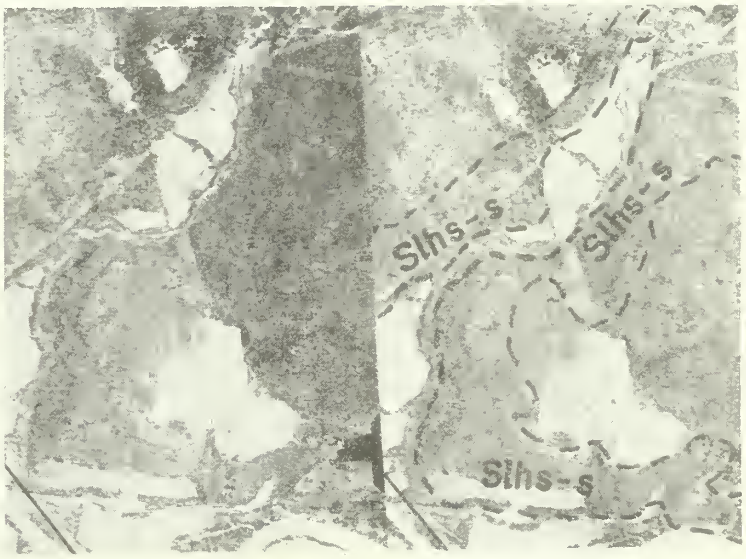

Figure 9.—Airphoto illustrating map unit Slhs-s.

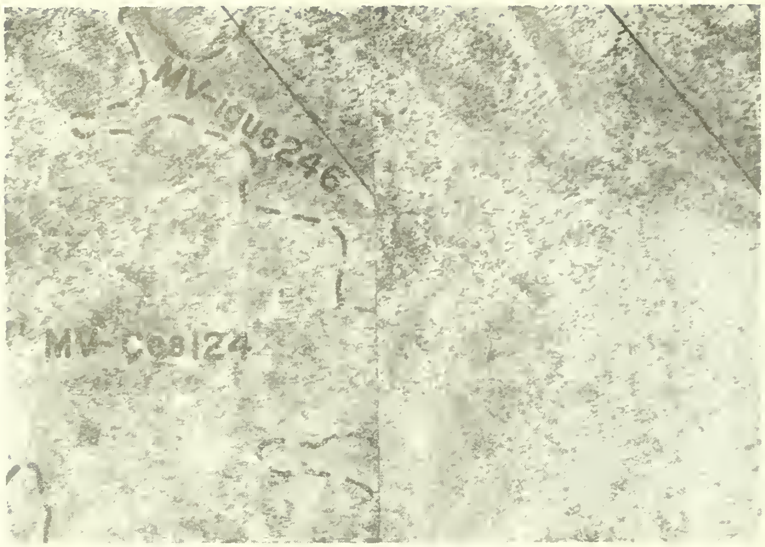

Figure 10.—Airphoto illustrating map units MV-Ces 124 *and* MV-igus 246.

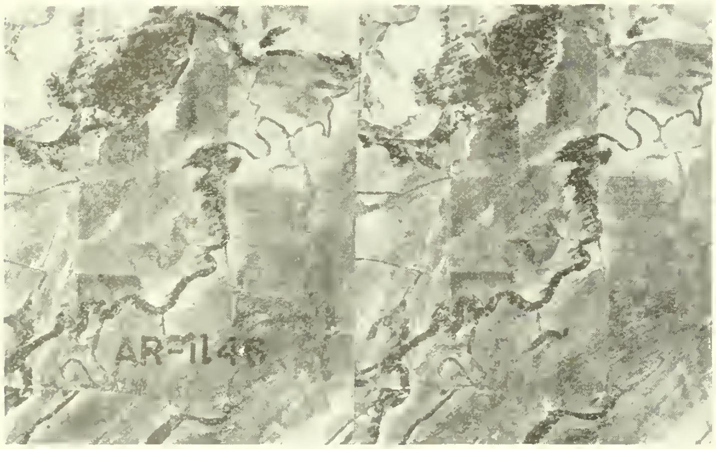

Figure 11.—Airphoto illustrating map unit AR-il 46.

higher unit onto residual soil derived from limestone, shale, or sandstone. Slopes usually range from 10 to 30 percent. The wash material varies from a few feet to about 12 feet in depth and may be A-2, A-4, or A-6 soil. Underlying the wash material is usually A-6 soil but it may be A-7. Internal drainage is usually good in the wash material, but downward movement of ground water will be retarded by the plastic residual soil. In some areas the unit is deeply dissected. Deep cuts involve some excavation in the plastic residual soil.

Units on recent alluvium

AR-gl 46.—This inextensive unit occupies flood plains which are rarely actually flooded. The soils are well-drained, friable, A-4 or A-6, and may contain some sand or gravel, particularly along existing or abandoned stream channels. The airphoto color tone is uniform dull gray.

AR-il 46.—The airphoto pattern of this unit is shown in figure 11. The unit occupies flood plains of which portions are normally flooded each year and other portions less frequently; all portions have a high ground-water table during other significant periods. Soils are plastic A-4 or A-6 but the profile may contain some strata of fine silty sand. Some areas have abandoned channels which contain organic clay. Airphoto color tone is dull gray mottled with darker gray, indicating silt or silty clay.

AR-pl 46.—The unit may be flooded each year and has a permanent high ground-water table. The soil is normally a plastic A-4 or A-6 soil but may have strata of silty sand. Depressed, wooded areas and abandoned channels contain organic clay. The airphoto color tone is medium to dark gray mottled with darker gray or black.

AR-gl 2.—The airphoto pattern of this unit is shown in figure 12. The unit occurs primarily as a natural levee adjacent to a stream and has a high water table only during flood stages, but is normally well-drained. Although the material is usually A-2 soil, there may be considerable variation in texture both horizontally and vertically, and some strata may contain gravelly sand or sandy gravel. The topography is nearly level but may have some shallow stream markings.

AR-il 2.—The unit occurs in either a natural levee of a stream or on the flatter portion of the flood plain, may be flooded more than once each year, and has a relatively high ground-water table during significant portions of each year. In general, the soil is A-2, but there may be thin strata of either A-4 or A-6 soil. Some areas have abandoned channels which may contain organic clay. Higher areas of the unit have a light gray to white airphoto color tone.

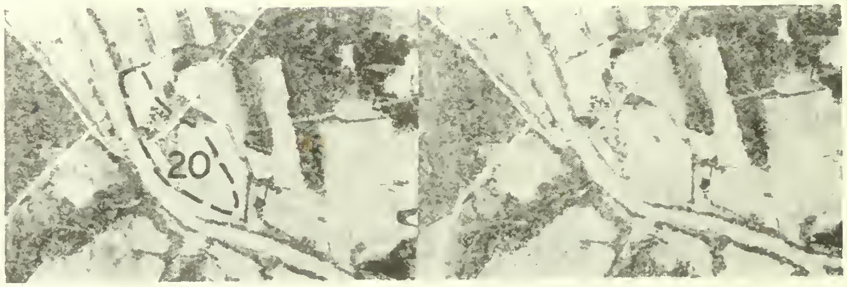

Figure 12.—Airphoto illustrating map unit AR-gl 2.

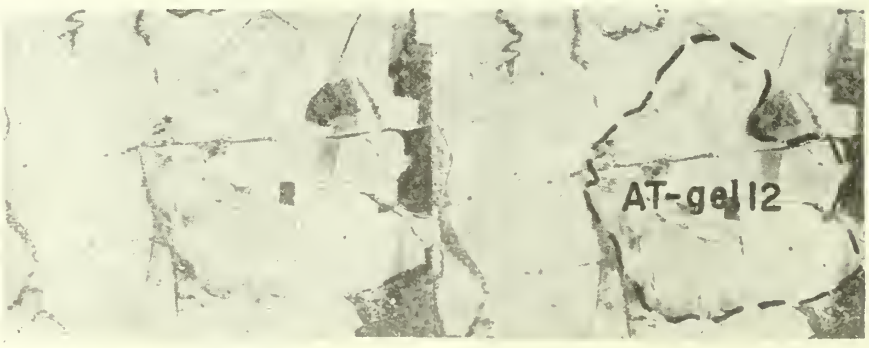

Figure 13.—Airphoto illustrating map unit AT-gel 12.

Units on alluvial terraces

AT-gel 12.—The airphoto pattern of this unit is shown in figure 13. The unit occupies slightly elevated positions along major streams and is rarely flooded; its ground surface varies from flat to gently undulating. There is little surface drainage; the outer terrace wall is steep, dissected by short, steep-walled gullies. The B-horizon soil may be A-4 or A-6 with some gravel, but the underlying soil is usually A-1 or A-2 with considerable clay and silt in the upper portion. Extreme vertical variation in textural gradation should be anticipated. Use as base course material is questionable, but some deposits should be good sources of borrow for use in adjacent embankments crossing poorly drained alluvial soils. The over-all medium gray color tone indicates that the surface soil may be gravelly clay, while the gray mottled areas have a thicker surfacing of clayey soil.

AT-gl 24.—The airphoto pattern of this unit is shown in figure 14. The unit occupies a position about 15 to 30 feet higher than the adjacent flood plain and the soils are usually friable, well-drained A-2 or A-4. Streams from the upland continue across the unit but surface drainage does not usually develop on the terrace. The light gray color tone with only minor mottling indicates that the material is sand. It may be a good source of borrow.

AT-il 6.—The unit occupies level to gently undulating areas 20 to 75 feet above the adjacent flood plain. The soil is usually A-6 but in some areas the B-horizon has A-7 soil and substrata are A-4. Internal drainage is imperfect and extensive surface drainage channels have developed in some areas. It may have a hardpan at a depth of 2 to 5 feet. In addition to the foregoing features, airphotos show the terrace has a gently sloping outer terrace wall (except where there has been recent stream cutting) and the color tone is dull gray mottled with darker gray.

Adequacy of Symbol System

It is believed that the system of map unit symbols proposed by Lueder and modified in this report adequately describe the most important soil profile and environmental characteristics of the area investigated, and that the coding system can readily be mastered since most of the symbols are suggestive of particular characteristics of map units. In preliminary location work the highway engineer is primarily interested in topographic details, with less consideration being given to soil conditions. In design and construction phases, the engineer is interested in topographic details, but soil characteristics, geology, and drainage conditions are also of primary importance. The symbols denote these characteristics in a generalized manner, while the detailed descriptions of map units enlarge upon the characteristics.

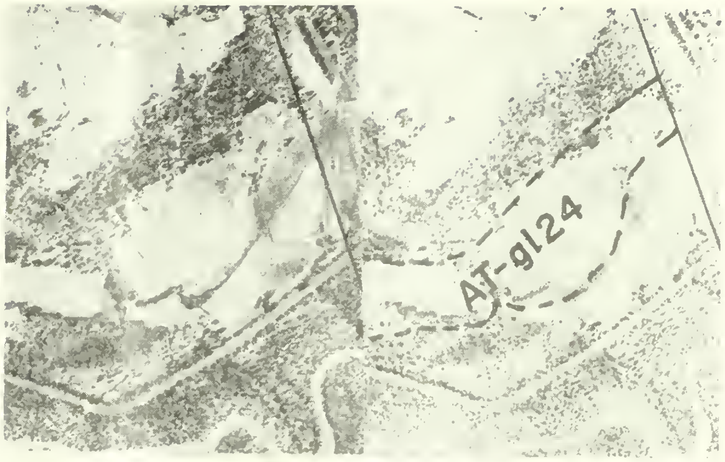

Figure 14.—Airphoto illustrating map unit AT-gl 24.

Adequacy of the mapping for the Natchez Trace Parkway will only be determined when the maps are used. If it is determined that more generalized mapping will suffice for future projects, the generalization can be done either by combining two or more symbols used to describe a particular characteristic of a unit or by eliminating one or more of the characteristics from the map unit designation. More detailed mapping could be accomplished by delineating smaller areas as map units and by using additional symbols to describe other characteristics of the map unit.

Geologic and agricultural soil survey units corresponding to the engineering soil map units delineated in section 2-D are listed in table 2. In general, the engineering map is more detailed than the geologic map. In agricultural soil mapping, the surface soil is usually of the greatest importance, whereas the engineer is also interested in lower soil horizons. An engineering map unit may be either a subdivision or combination of agricultural soil types.

Design and Construction Problems

Many problems of highway design and construction can only be solved by field study or application of previously obtained knowledge concerning a similar problem. Information relating to problems which are characteristic of individual map units has been noted in the descriptions of the engineering characteristics of the units. Engineeding information concerning characteristics common to several map units is presented in table 3.

Frost does not penetrate the soils to a great depth, and below-freezing periods are of relatively short duration, in the area under consideration; hence, frozen soil is not a major deterrent to earthwork. However, there is considerable rainfall during winter and early spring months, which delays construction in granular soils and prevents grading operations in fine-grained soils for substantial periods.

In some map units, particularly in limestone areas, the depth to ground-water table at any time varies considerably over relatively small areas. When the water table is deep it usually has no influence on highway construction, and when at a moderate depth it is of consequence only in deep cuts. The designation "shallow" in table 3 indicates that the water table may be at or near the ground surface for signicant periods; "high" indicates those areas which may be inundated for significant periods, with the water table near the ground surface during the remainder of the year.

In any map unit the road surface should be at such an elevation that the pavement will not be adversely affected by the ground water table. In alluvial map units, the

Table 2.—*Correlation of engineering map units with geologic materials and agricultural soils*

Engineering map unit	Geologic material	Agricultural soil type
Sl-Ciglu 6	Cherty limestone	Dewey loam.
SlV-piu 6	do	Do.
SlV-pis 6	do	Do.
SlV-s	do	Rough stony land.
ShV-pius 67	Limestone, calcareous shale, and sandstone	Colbert clay or silt loam.
Slhs-s	do	Colbert clay.
SsV-els 234	Sandstone	Atwood gravelly loam and Dickson silt loam.
Mv-Ces 124	Tuscaloosa formation (gravel, sand, and clay)	Atwood gravelly loam and Guin undifferentiated.
MV-igus 246	do	Guin undifferentiated.
AR-gl 46	Recent alluvium	Ochlockonee fine sandy loam.
AR-il 46	do	Ochlockonee fine sandy loam or silt loam and Huntington silt loam.
AR-pl 46	do	Huntington silt loam.
AR-gl 2	do	Ochlockonee fine sandy loam.
AR-il 2	do	Do.
AT-gel 12	do	Cahaba gravelly fine sandy loam.
AT-gl 24	do	Kalmia fine sandy loam.
AT-il 6	do	Kalmia loamy fine sand.

Table 3.—*Engineering data and recommendations for map units*

Map unit designation	Adapted to winter grading	Normal position of water table	Location of grade line with respect to ground surface	Erosion resistance	Possible source of—		
					Borrow	Granular subbase material	Base course material
Sl-Cglu 6	No	Variable	Shallow cuts in ground swells; fills in depressions.	Good	Limited	No	No
SlV-pnu 6	No	Shallow	do	do	No	No	No
SlV-pns 6	No	Variable	Influenced by bedrock	do	No	No	No
SlV-s	Limited	Deep	do	do	No	No	Yes
SsV-els 234	Yes	do	Anywhere	do	Yes	Limited	No
ShV-pnus 67	No	Variable	do	Fair to poor	No	No	No
Slhs-s	Limited	do	Influenced by bedrock	Good to fair	No	No	Yes
MV-Ces 124	Yes	Deep	Anywhere	Good	Yes	Yes	Yes
MV-agus 246	Limited	Variable	do	Fair	Yes	Limited	No
AR-gl 46	No	Moderate	Above high water		Limited	No	No
AR-il 46	No	Shallow	do		No	No	No
AR-pl 46	No	High	do		No	No	No
AR-gl 2	Yes	Moderate	do		Yes	Yes	No
AR-il 2	No	Shallow	do		Limited	Limited	No
AT-gel 12	Yes	Moderate to deep.	Anywhere, if above high water.	Good	Yes	Yes	Yes
AT-gl 24	Yes	do	Anywhere	do	Yes	Limited	No
AT-il 6	No	do	do	do	No	No	No

grade line should be sufficiently above high water elevation to prevent damage to the pavement.

In those units designated as limited sources of borrow or subbase material, the soil is either of such poor quality that it should be used only when better material cannot be obtained, or it is difficult to excavate because of high ground-water table during significant periods.

Since there are extensive sources of gravel in the mapped area, it is probable that a pavement section composed of gravel base course and bituminous concrete surface course will be adopted for section 2-D. However, the normal base course thickness might be reduced in those areas where the subgrade is composed of granular soils, whereas an above-normal thickness of pavement may be required in areas where the subgrade is composed of very plastic soils. In some areas the necessity for a thicker base course may be alleviated by topping the subgrade with one of the materials designated as granular subbase material in table 3.

Parent formations, as well as unconsolidated materials, are represented in table 3 as sources of base course material, although it is probable that gravel will be used exclusively. Map unit *MV-Ces 124* will probably be the principal source of gravel for section 2-D, but considerable exploration may be required in order to find a deposit of gravel suitable for a base course. The terrace deposit *AT-gel 12* may have some suitable gravel, but it is limited in extent.

Time Required for Preparation of Report

So that the information obtained on the Natchez Trace study may be used in planning other mapping projects, the estimated time required, in 8-hour man-days per square mile of mapped area, for various phases of the investigation of section 2-D is given in table 4. The table does not show the amount of time required for map drafting, stenographic work, and preparation of the report, because each of those items will depend upon the use to be made of the report.

It is believed that the developed system of map-unit symbols is adequate for describing the major soil and terrain characteristics of an area; hence, any future mapping will not require any time for that phase of the investigation.

The time required for the office study of published material, correlation of airphoto patterns with soils and geology, and airphoto delineation of map unit boundaries depends upon (1) the amount of published material which must be read in order to obtain the basic information on soils and geology, (2) complexity of mapping, and (3) ability of the airphoto interpreter. Complexity of mapping depends not only upon the variations in geologic materials and environmental factors but also the purpose of the mapping. A highly generalized map may suffice for the initial highway reconnaissance survey, whereas a more detailed map will be required in a later stage of highway location.

The skill of the airphoto interpreter depends not only upon his ability to associate various features of airphoto patterns but also his experience in mapping similar areas. After the airphoto pattern for a particular map unit has been defined, airphoto delineation of the boundary between an area of that map unit and adjacent map units is usually not difficult. Mapping of a strip adjacent to a previously mapped area might require only half as much time per unit of area, because most of the map units would be repeated in the second strip.

The time required for the field survey and soil testing depends upon (1) number of map units, (2) stratification of the parent material, (3) topography, (4) amount of information previously obtained for the same map units, and (5) ability of the surveyor to select sampling sites which are representative of the particular map unit. Ordinarily, for any coastal plain map unit several strata will be encountered within the normal depth of highway excavation, hence several samples will be required in order to determine the characteristics of the various soils. On the other hand, in a residual soil map unit, the parent material is not usually stratified, and less sampling will be required. Borings in the areas having steep slopes or irregular topography must usually be deeper than in flat areas; hence, more time will be required in the field soil survey of the steep areas. It is probable that, because of the variety of parent materials encountered both laterally and vertically and the irregular topography, the 35 samples obtained in section 2-D are more than would normally be required in an area of that size. If the only tests required are those for the Highway Research Board classification, the time required for soil testing will be less than the 1.0 man-day per square mile estimated for section 2-D.

Frequently, the information regarding map units in one area can be applied to either the same or similar map units in another area, thus reducing the amount of field work in the second area.

The time required for airphoto delineation of drainage lines varies according to the intricacy of dissection of the area, so it is probable that less than 0.05 man-day would be required to delineate the drainage in a square mile of a slightly or moderately dissected area. A highly trained airphoto interpreter is not required for airphoto delineation of drainage lines.

The primary aim of the engineering report is to present information which can be used by the highway engineer; hence, a considerable portion of the total time requirement should be allotted for map unit description and engineering recommendations. The engineer who makes the recommendations should have a broad knowledge of the factors involved in highway location, design, and construction. He should be familiar with construction practices in either adjacent areas or other areas having similar soil and environmental characteristics. For each map unit he must consider the possible uses to be made of the mapping, and then present engineering in-

Table 4.—*Time required for preparation of engineering report*

Phase of investigation	Time required, man-days per square mile	
	Engineer-interpreter	Other personnel
Office study of soil bulletins and geologic reports	0.15	0.00
Development of system of map symbols	0.05	0.00
Preliminary study of airphotos and correlation of airphoto patterns with soils and geology	0.25	0.10
Airphoto delineation of map unit boundaries	0.20	0.00
Field survey	0.15	0.15
Soil testing	0.00	1.00
Drainage	0.05	0.00
Map unit description and engineering recommendations	0.15	0.00
Total	1.00	1.25

formation which will be of value to the user.

The total of 2.25 man-days per square mile required for the mapping and reporting of section 2-D, as shown in table 4, is greater than for adjacent construction sections of the parkway because section 2-D has a greater variation in geologic formations and topography. Complexity of geologic formations and irregular topography require the expenditure of a great amount of time for mapping, particularly when the mapped strip is alined approximately in the direction of dip, as it is for section 2-D. It is probable that not more than 1.5 man-days per square mile would be required to map a mile-wide strip in a slightly to moderately dissected area having extensive exposures of relatively uniform geologic formations, and that time might be considerably reduced if the airphoto interpreter had mapped other areas having the same or similar airphoto patterns. The time would be about equally divided between the engineer-interpreter and other personnel. For a wider strip the average time required for mapping a square mile might be reduced to 1.0 man-day.

Inter-American Highway Film

Inter-American Highway Report—Part 1, Mexico, a motion picture produced by the Bureau of Public Roads, is now available on loan to interested organizations. The 16-millimeter sound and color film, with a running time of 55 minutes, shows the present condition of the Mexican portion of the Inter-American Highway from Laredo, Texas, to the Guatemalan border—1,700 miles of excellent roads built by the Republic of Mexico with its own funds and personnel.

The scenes picture a realm of contrasts in a setting of magnificent scenery. There are thriving cities and relics of antiquity, primitive and modern means of transportation, native handicraft and up-to-date industrial plants, antiquated and scientifically operated farms, and many other colorful and interesting sights to capture the attention of the traveller. Typical hotel, garage, and filling-station services are shown. Animated maps represent the locations of many of the scenes depicted.

Inter-American Highway Report—Part 1, Mexico may be borrowed by any responsible organization, without cost except for nominal transportation charges, by writing to the Visual Education Branch, Bureau of Public Roads, Washington 25, D. C.

Work on a companion film is now in progress. This second installment will show the condition of the continuation of the Inter-American Highway through the six Central American republics, terminating at Panama City.

Freezing and Thawing Tests of Concrete Containing Oregon and Washington Aggregates

Reported by A. G. TIMMS
Senior Materials Engineer

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

In this study, plain and air-entrained concretes made with certain Oregon and Washington aggregates, and with Potomac River aggregates as a control, were subjected to alternate freezing and thawing after preliminary moist curing of both short and prolonged periods. Some of the concretes had poor resistance to freezing and thawing while others stood up well. There was no evidence in most cases of any alkali-aggregate reaction.

Prolonged initial curing improved the resistance to freezing and thawing of all but one of the non-air-entrained concretes. This was not true for the air-entrained concretes, and in many cases the additional curing was actually harmful. However, the air-entrained concretes with prolonged initial curing generally had better resistance to freezing and thawing than the comparable non-air-entrained concretes cured for only a short time.

In all cases, when the preliminary curing period was short, the use of an air-entraining agent produced concrete with greater resistance to freezing and thawing. With the prolonged preliminary curing, air entrainment was not beneficial to three of the five aggregate combinations.

In general, the tests indicated that the durability of air-entrained concrete may be adversely affected by conditions which involve long-continued exposure to moisture. It was also found that the results of freezing and thawing tests made after a short period of curing in moist air may not be sufficient to evaluate properly the durability of concrete containing certain cement-aggregate combinations.

IN a paper published in 1944[1] Bailey Tremper, Materials Engineer of the Washington State Department of Highways, presented data showing that concrete containing aggregates from certain sources in eastern Washington, in combination with cements high in alkalies, when tested in freezing and thawing after prolonged preliminary moist curing (7 months), broke down quite rapidly as compared to similar concretes containing either the same aggregates in combination with low-alkali cements or neutral aggregates in combination with high-alkali cements. However, he found that when these same combinations were subjected to relatively short preliminary moist curing (28 days), all of them developed about the same resistance to freezing and thawing. He also noted that 1- by 1- by 10-inch mortar bars containing these combinations, when exposed in a saturated atmosphere at 70° F., failed to develop any significant expansion at ages up to 4 years. On the other hand, when these mortar bars were subsequently frozen and thawed, the order of deterioration followed in general the same pattern as noted for the concrete.

Tremper concluded from these data that the lack of durability observed in his tests was primarily a function of the alkali content of the cement. However, he believed that the type of reaction was distinctly different from that which had previously been reported by Stanton and other investigators of the so-called "alkali-aggregate" reaction.

The tests reported here were made to obtain additional information on this particular point as well as to study the durability of concrete made with certain other cement-aggregate combinations commonly used in Oregon and Washington.

Seventeen combinations of cements and aggregates in concrete were investigated. The materials included four fine aggregates, four coarse aggregates, and three cements from the Washington-Oregon area, Potomac River sand and gravel from the vicinity of Washington, D. C., and two cements manufactured in the Southeast, one having a high and the other a low alkali content. The combinations involving Oregon and Washington materials were selected on the basis of recommendations by Tremper and by G. S. Paxson, Bridge Engineer, Oregon Highway Commission. The combinations of Potomac River sand and gravel with the various cements were tested to provide a comparison between the Oregon and Washington materials and combinations in which the same cements were used with an eastern aggregate. Concrete specimens containing the 17 combinations of materials were tested for resistance to alternate freezing and thawing in water. Two series of tests were made—one in which the freezing cycle was started at the expiration of 28 days of preliminary moist curing, and the other in which freezing was started at the conclusion of 180 days of moist curing. These periods were selected in order to bring out any difference in behavior which might be due to alkali action under long-continued moist curing, as observed by Tremper.

Two parallel series of tests were made—one in which the plain portland cements were used, and the other in which sufficient neutralized Vinsol resin was added at the mixer to give an air content of approximately 4 percent in the plastic concrete.

Conclusions

The following conclusions appear warranted from the data:

1. In only two of the seventeen combinations was there evidence of a probable alkali-aggregate reaction. These were the combinations involving the two relatively high-alkali cements which were used with the sand and gravel from Rock Island, Wash. This tendency was noted in the case of both the non-air-entrained and the air-entrained concrete.

2. Non-air-entrained concrete containing the sand from Mt. Hebron, Calif., in combination with the crushed stone from Klamath Falls, Oreg., developed very poor resistance to the action of alternate freezing and thawing with all three of the cements used with these aggregates. However, no tests were made to determine the contributing factor of either aggregate, when used separately, to the poor resistance of the combination.

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[1] *The effect of alkalies in portland cement on the durability of concrete*, by Bailey Tremper. Journal of the American Concrete Institute, November 1944.

Table 1.—*Properties of the cements*

		Cement				
		1	2	3	4	5
Physical properties:						
Apparent specific gravity		3.17	3.13	3.11	3.14	3.12
Specific surface	cm.^{2} per gm.	1,810	1,810	1,760	1,880	1,840
Autoclave expansion	percent	0.03	0.04	0	0.01	0.21
Normal consistency	percent	24.5	24.5	24.5	25.5	25.0
Time of set:						
Initial	hr: min.	4:30	4:10	3:00	5:10	3:45
Final	hr: min.	6:00	6:00	5:45	8:15	6:15
Tensile strength:						
At 3 days	lb. per sq. in.	280	275	270	280	280
At 7 days	lb. per sq. in.	365	365	310	355	375
At 28 days	lb. per sq. in.	445	450	430	485	435
Sugar test:						
Neutral point	ml.	4.5	28.7	43.3	2.8	3.9
Clear point	ml.	4.7	38.9	60.1	2.8	4.0
Air in mortar [1]	percent	4.5	4.7	4.6	2.5	7.6
Chemical analyses (percent)						
Silica (SiO_2)		20.90	21.25	22.30	22.60	21.75
Alumina (Al_2O_3)		4.33	5.30	4.38	4.30	4.79
Ferric oxide (Fe_2O_3)		5.12	3.60	3.52	4.40	2.96
Calcium oxide (CaO)		64.45	63.70	65.30	64.05	63.20
Magnesia (MgO)		2.25	1.76	0.91	1.25	2.48
Sulfuric anhydride (SO_3)		1.65	1.66	1.73	1.68	1.84
Sodium oxide (Na_2O)		0.35	0.20	0.08	0.31	0.05
Potassium oxide (K_2O)		0.17	0.47	0.15	0.19	0.93
Loss on ignition		0.90	2.04	1.62	1.17	2.11
Chloroform soluble material [2]		0.005	0.013	0.006	0.015	0.004
Na_2O equivalent		0.46	0.51	0.18	0.44	0.66
Computed compound composition (percent): [3]						
Tricalcium silicate (C_3S)		61	52	57	49	50
Dicalcium silicate (C_2S)		14	21	21	28	25
Tricalcium aluminate (C_3A)		3	8	6	4	8
Tetracalcium alumino-ferrite (C_4AF)		16	11	11	13	9
Calcium sulfate ($CaSO_4$)		3	3	3	3	3

[1] A.S.T.M. method C 185-44T

[2] A.S.T.M. method C 114-42.

[3] The compound compositions given are in "shorthand" form.

3. Both non-air-entrained and air-entrained concrete containing the sand and gravel from Umatilla, Oreg., and the sand and gravel from the Willamette River, Oreg., had good resistance to alternate freezing and thawing with the same three cements as were used with the Mt. Hebron-Klamath Falls aggregate combination.

4. Non-air-entrained concrete made with Potomac River sand and gravel had very poor resistance to freezing and thawing with all five cements, but there was no apparent indication of an alkali-aggregate reaction.

5. In the case of the non-air-entrained concrete, increasing the time of initial curing improved resistance to freezing and thawing for all combinations except that involving the Rock Island aggregates and the cement with the highest alkali content. In the case of the air-entrained concrete, increasing the curing time did not in general increase resistance to alternate freezing and thawing and in many cases the additional curing was harmful. However, in only one case (the Rock Island aggregate with cement 5) was the resistance of air-entrained concrete, cured for 180 days, less than that of the comparable non-air-entrained concrete cured for 28 days.

6. In all cases, for concrete cured 28 days, the use of an air-entraining agent was beneficial in producing concrete with greater resistance to freezing and thawing than was developed by the corresponding concrete without air entrainment. For concrete cured 180 days, air entrainment was definitely beneficial only in the case of combinations involving the Mt. Hebron sand with the Klamath Falls coarse aggregate and the Potomac River sand and gravel. Under these conditions (180 days of curing), air entrainment did not appreciably improve the resistance of combinations containing the other aggregates. In general, improvement due to air entrainment was much less marked after 180 days of curing than after 28 days.

In general, the tests indicated that the durability of air-entrained concrete may be adversely affected by conditions which involve long-continued exposure to moisture. The results of freezing and thawing tests made after 28 days of curing in moist air may not be sufficient to evaluate properly the durability of concrete containing certain cement-aggregate combinations.

Sources of Materials

Five portland cements were used in this investigation, three from the West Coast and two from the Southeast. One of the southeastern cements was selected for its low total alkali content, the other because it contained a relatively high percentage of alkali, mostly potash. The physical properties, chemical analyses, and computed compound compositions are given in table 1. All cements complied with the standard

Table 2.—*Properties of the fine aggregates*

		Mt. Hebron, Calif.	Umatilla Oreg.	Willamette River, Oreg.	Potomac River, Md.	Rock Island, Wash.
Grading: percentage passing—						
No. 4 sieve		96	95	91	98	98
No. 8 sieve		77	86	71	85	84
No. 16 sieve		60	79	59	71	67
No. 30 sieve		47	50	42	49	41
No. 50 sieve		30	10	14	18	10
No. 100 sieve		10	2	2	4	2
No. 200 sieve		3.3	0.8	0.6	1.8	0.6
Fineness modulus		2.80	2.78	3.21	2.75	2.98
Bulk specific gravity:						
Dry		2.42	2.67	2.51	2.59	2.63
Saturated surface-dry		2.51	2.73	2.58	2.63	2.66
24-hour absorption	percent	3.9	2.2	2.9	1.6	1.0
Sodium sulfate test: loss at 5 cycles	percent	10.5	5.2	6.5	----------	4.1
Compressive strength ratio: [1]						
At 7 days		1.18	1.04	87	1.13	9.8
At 28 days		.98	1.09	1.01	1.16	9.7
Tensile strength ratio: [1]						
At 7 days		.97	1.13	1.09	1.17	1.18
At 28 days		.88	1.15	1.04	1.14	1.21

[1] A.A.S.H.O. T 35-35.

Table 3.—*Properties of the coarse aggregates*

		Klamath Falls, Oreg., crushed basalt	Umatilla, Oreg., gravel	Willamette River, Oreg., gravel	Potomac River, Md., gravel	Rock Island, Wash., gravel
Grading: percentage passing—						
1-inch sieve		100	100	100	100	100
¾-inch sieve		94	64	83	75	69
½-inch sieve		50	28	44	40	42
⅜-inch sieve		28	11	13	20	10
No. 4 sieve		0	1	0	0	0
Fineness modulus		6.78	7.25	7.04	7.05	7.21
Bulk specific gravity:						
Dry		2.71	2.72	2.58	2.56	2.68
Saturated surface-dry		2.75	2.74	2.63	2.59	2.69
24-hour absorption	percent	1.3	0.9	2.1	1.1	0.7
Sodium sulfate test: loss at 5 cycles	percent	2.5	1.3	8.8	0.9	1.8
Los Angeles abrasion loss [1]	percent	17.6	14.6	16.4	30.3	14.6
Weight per cu. ft., dry rodded	lb.	92	104	105	107	111

[1] Grading B.

Table 4.—*Expansion of 1:2 mortar bars, arranged in ascending order of values of alkali content of cements*

Aggregates	Expansion of mortar bars (in percent) after—									
	Moist storage at 120° F. for 3 years, using cement— [1]					Moist storage at 120° F. for 12 weeks, using cement— [1] (1 percent [2] NaOH added)				
	3 (0.18)	4 (0.44)	1 (0.46)	2 (0.51)	5 (0.66)	3 (0.18)	4 (0.44)	1 (0.46)	2 (0.51)	5 (0.66)
Mt. Hebron sand	−0.01	----	0.01	0.01	----	0.01	----	0.03	0.04	----
Klamath Falls basalt	.02	---	.13	.13	---	.04	---	.17	.13	---
Umatilla gravel	.01	---	.04	.03	---	.03	---	.09	.10	---
Umatilla sand	.01	---	.02	.05	---	.02	---	.07	.08	---
Willamette River gravel	−.01	---	.04	.02	---	.00	---	.06	.12	---
Willamette River sand	−.01	---	.02	.01	---	.04	---	.16	.12	---
Rock Island gravel	.02	0.08	---	---	0.12	.04	0.09	---	---	0.10
Rock Island sand	.00	.05	---	---	.08	.03	.19	---	---	.18
Potomac River gravel	.01	.02	.02	.03	.05	.03	.05	.06	.06	.09
Potomac River sand	.01	.02	.03	.03	.06	.02	.05	.08	.10	.09

[1] Alkali content, indicated in parentheses under the cement numbers, = percentage $Na_2O + 0.658\ K_2O$.
[2] By weight of cement.

Table 5.—*Cement-aggregate combinations*

Aggregate combination No.	Cements	Fine aggregate source	Coarse aggregate source
I	1, 2, 3	Mt. Hebron, Calif.	Klamath Falls, Oreg.
II	1, 2, 3	Umatilla, Oreg.	Umatilla, Oreg.
III	1, 2, 3	Willamette River, Oreg.	Willamette River, Oreg.
IV	1, 2, 3, 4, 5	Potomac River, Md.	Potomac River, Md.
V	3, 4, 5	Rock Island, Wash.	Rock Island, Wash.

specifications for type I portland cement of the American Society for Testing Materials. In addition, cements 4 and 5 also met the requirements of type II cements. Cements 1, 2, and 3, while complying with type II requirements with respect to the percentage of tricalcium aluminate, cannot be classified as type II because, in each case, the amount of tricalcium silicate exceeded 50 percent.

Five fine aggregates and five coarse aggregates were used. Four of each were from the Washington-Oregon area, and Potomac River sand and gravel were used as a basis of comparison. The sources as well as the gradings and other physical properties of the fine and coarse aggregates are given in tables 2 and 3 respectively. Nothing unusual in the properties of the aggregates is apparent from a study of the values in these tables, except that the sand from Mt. Hebron, Calif., appeared to be of relatively poor structural quality, as revealed by the comparatively high sodium sulfate soundness loss and the low tensile strength of the mortar.

The petrographic composition of the fine and coarse aggregates is as follows:

Mt. Hebron sand. — Rounded rhyolite, basalt, andesite, and felsite with quartz and magnetite.

Klamath Falls basalt.—Olivine basalt.

Umatilla gravel.—Angular and subangular granite, basalt, rhyolite, quartz, and quartzite.

Umatilla sand.—Angular and subangular basalt, rhyolite, and felsite, with some granite, quartz, and limestone.

Willamette River gravel. — Rounded basalt, andesite, rhyolite, and felsite.

Williamette River sand. — Angular and subangular rhyolite and felsite, with some basalt, granite, and quartz.

Rock Island gravel. — Rounded granite, syenite, rhyolite, and basalt.

Rock Island sand. — Angular and subangular granite, rhyolite, felsite, and basalt, with some feldspar and quartz.

Potomac River gravel.—Rounded quartz, quartzite, chert, and sandstone, with some granite, gneiss, and schist.

Potomac River sand.—Subangular quartz, quartzite, and chert, with some sandstone, schist, granite, and magnetite.

Mortar-Bar Tests

The results of expansion tests of 1:2 mortar bars are given in table 4. In making the mortar bars, the coarse aggregates were crushed so that 100 percent passed the No. 4 sieve and 90 percent passed the No. 8 sieve. Only those particles of the sands passing the No. 4 sieve were used in the test. The coarse aggregates and the sands were each tested separately. Sufficient water was used with the 1:2 mortar to give a plastic consistency. The specimens were 1- by 1- by 11½-inches in size.

One group of specimens was prepared with plain mortar, and a second group with

Table 6.—*Summary of mix data for non-air-entrained concrete*[1]

Aggregate combination and cement	Proportions of mix, by oven-dry weight	Cement content	Net water content	Slump	Weight of fresh concrete	Calculated air content
	Pounds	*Sacks per cu. yd.*	*Gal. per sack*	*Inches*	*Lb. per cu. ft.*	*Percent*
I-1	94:212:241	6.4	6.4	3.2	145.7	1.0
I-2	94:212:241	6.4	6.4	2.9	145.4	1.1
I-3	94:212:241	6.4	6.4	3.3	145.3	1.0
II-1	94:188:300	6.5	5.4	3.2	152.9	1.3
II-2	94:188:300	6.5	5.4	2.9	152.4	1.3
II-3	94:188:300	6.5	5.4	2.8	152.5	1.4
III-1	94:198:272	6.5	5.4	3.0	148.4	0.7
III-2	94:198:272	6.4	5.3	3.0	148.1	0.9
III-3	94:198:272	6.4	5.4	3.0	148.1	0.6
IV-1	94:192:278	6.4	5.5	3.1	146.9	1.1
IV-2	94:192:278	6.4	5.6	3.0	146.5	1.2
IV-3	94:192:278	6.4	5.6	2.9	146.3	1.2
IV-4	94:192:278	6.4	5.6	2.8	146.8	1.1
IV-5	94:192:278	6.4	5.5	3.2	146.7	1.2
V-3	94:215:264	6.5	5.3	2.8	149.3	2.0
V-4	94:215:264	6.5	5.3	3.0	150.0	1.4
V-5	94:215:264	6.5	5.3	3.3	149.5	1.8

[1] Each value is average of three tests.

Table 7.—*Summary of mix data for air-entrained concrete*[1]

Aggregate combination and cement	Proportions of mix, by oven-dry weight	Vinsol-resin added [2]	Cement content	Net water content	Slump	Weight of fresh concrete	Calculated air content
	Pounds	*Percent*	*Sacks per cu. yd.*	*Gal. per sack*	*Inches*	*Lb. per cu. ft.*	*Percent*
I-1	94:198:241	0.0087	6.5	6.0	2.9	142.1	3.9
I-2	94:198:241	.0095	6.4	6.0	3.0	141.3	4.3
I-3	94:198:241	.0112	6.4	6.0	3.0	141.5	4.1
II-1	94:173:300	.0078	6.5	5.0	2.8	149.1	4.3
II-2	94:173:300	.0085	6.5	5.1	3.0	148.2	4.6
II-3	94:173:300	.0103	6.5	5.1	3.0	148.6	4.3
III-1	94:184:272	.0080	6.5	5.0	3.2	145.3	3.3
III-2	94:184:272	.0092	6.5	5.0	2.8	144.7	3.6
III-3	94:184:272	.0105	6.4	5.0	3.0	144.1	3.9
IV-1	94:178:278	.0072	6.5	5.2	3.0	143.3	4.1
IV-2	94:178:278	.0075	6.4	5.2	3.0	142.5	4.4
IV-3	94:178:278	.0098	6.4	5.2	3.1	142.6	4.3
IV-4	94:178:278	.0097	6.4	5.3	3.1	143.1	4.0
IV-5	94:178:278	.0075	6.5	5.0	3.2	143.3	4.3
V-3	94:201:264	.0102	6.5	5.0	3.2	145.5	4.8
V-4	94:201:264	.0098	6.5	5.0	3.2	146.3	4.4
V-5	94:201:264	.0072	6.5	4.8	3.2	145.5	5.2

[1] Each value is average of three tests.
[2] Percentage by weight of cement, added as a sodium hydroxide solution.

Table 8.—*Decrease in resistance of concrete to freezing and thawing, after initial moist curing of 28 days, as measured by decrease in* N^2

Aggregate combination and cement	Percentage decrease in N^2 after — cycles of freezing and thawing [1]															Durability factor[2]
	8	16	24	32	40	48	56	64	72	80	88	96	104	112	120	
NON-AIR-ENTRAINED CONCRETE																
I-1	10	22	[3]42	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	14
I-2	22	41	[4]51	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	8
I-3	25	38	[4]45	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	9
II-1	6	6	7	8	9	9	9	10	12	14	15	16	16	13	14	86
II-2	6	6	7	9	12	14	14	14	16	18	20	21	22	20	20	80
II-3	5	6	6	9	10	12	15	16	20	24	26	26	29	31	39	61
III-1	4	3	3	5	5	5	6	7	9	9	9	11	12	9	9	91
III-2	4	4	6	8	9	9	12	12	15	16	16	18	20	17	18	82
III-3	4	4	6	8	12	15	21	23	25	27	29	31	32	33	33	67
IV-1	9	12	16	25	32	37	[5]40	-----	-----	-----	-----	-----	-----	-----	-----	26
IV-2	11	14	18	26	31	36	[5]40	-----	-----	-----	-----	-----	-----	-----	-----	26
IV-3	14	23	30	[6]46	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	14
IV-4	14	27	[7]40	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	12
IV-5	10	12	14	18	21	25	30	34	[8]44	-----	-----	-----	-----	-----	-----	35
V-3	5	5	4	5	5	5	5	5	7	9	9	13	17	14	14	86
V-4	6	6	7	8	9	11	12	13	20	22	24	27	32	33	33	67
V-5	8	8	8	9	8	9	9	10	11	12	13	13	15	14	15	85
AIR-ENTRAINED CONCRETE																
I-1	3	3	1	9	10	12	12	13	17	17	16	17	17	16	14	86
I-2	2	5	6	10	15	20	23	27	30	31	32	43	44	44	44	47
I-3	1	1	2	2	4	6	8	9	12	12	10	13	11	11	9	91
II-1	2	3	3	3	3	4	4	5	7	6	6	5	5	4	2	98
II-2	2	4	5	4	4	5	6	6	9	10	12	11	10	9	7	93
II-3	2	2	3	2	2	4	6	4	6	6	6	8	5	5	4	96
III-1	0	1	3	0	1	2	2	3	3	2	3	3	3	2	+1	100
III-2	1	2	3	2	3	3	4	6	7	7	7	7	7	7	6	94
III-3	0	0	0	+1	+1	0	+1	0	0	0	+1	1	+1	+1	3	97
IV-1	5	5	6	4	5	6	5	6	6	6	5	6	6	3	4	96
IV-2	5	5	6	5	7	8	8	11	11	12	12	14	13	11	10	90
IV-3	4	5	4	3	2	4	3	5	6	7	6	8	6	6	3	97
IV-4	4	4	5	4	5	5	5	6	7	7	7	7	7	5	4	96
IV-5	5	5	5	4	3	6	4	6	7	7	5	5	5	4	2	98
V-3	3	3	2	1	0	2	1	2	2	2	2	1	1	2	1	99
V-4	3	3	3	3	3	3	3	5	5	6	5	4	4	3	1	99
V-5	4	5	5	4	3	4	4	6	5	6	4	4	4	2	1	99

[1] Each value is the average of tests on three beams except for II-2 in the non-air-entrained group, and I-3 and II-3 in the air-entrained group, for which two beams were tested, and III-1 in the air-entrained group, for which one beam was tested.

[2] Method of determination similar to that described in Tests for Air-Entraining Admixtures for Concrete, A.S.T.M. designation C 233-49T.

[3] Final reading at 29 cycles.
[4] Final reading at 21 cycles.
[5] Final reading at 53 cycles.
[6] Final reading at 32 cycles.
[7] Final reading at 24 cycles.
[8] Final reading at 75 cycles.

mortar to which was added 1 percent of sodium hydroxide, by weight of cement. The addition of sodium hydroxide was for the purpose of accelerating the action and is rather common practice in work of this type. The first group was stored at 120° F. for 3 years, and the second group at 120° F. for 12 weeks. For each aggregate the expansion data are arranged in order of ascending values of alkali of the cements, expressed as sodium oxide.

None of the plain mortar bars developed expansions which would be indicative of a definite alkali-aggregate reaction. However, the specimens containing the Klamath Falls basalt with cements 1 and 2 and those containing the aggregates from Rock Island with cements 4 and 5 developed somewhat higher expansions at 3 years than did any of the others. The same general trends were indicated also in the case of the specimens containing 1 percent added sodium hydroxide and stored 12 weeks. The results of these tests would classify the Klamath Falls basalt and the Rock Island materials as mildly alkali reactive. However, as will be seen later, freezing and thawing tests on the concrete would indicate that the rapid failure of the concrete containing the basalt was due primarily to some other factor.

Test Procedures

The materials used in each of the 17 cement-aggregate combinations are shown in table 5. The Potomac River sand and gravel were the only aggregates used with all five cements. In general, each coarse aggregate was tested with the fine aggregate from the same general locality. As previously stated, the combinations involving Oregon and Washington materials were selected on the basis of recommendations by G. S. Paxson, Bridge Engineer of Oregon, and Bailey Tremper, Materials Engineer of Washington, respectively; the other combinations were for purposes of comparison.

Summaries of the concrete mix data for all combinations are given in tables 6 and 7. The nominal cement content was 6.5 sacks per cubic yard, and a slump of approximately 3 inches was obtained. The water content varied from a low of 4.8 gallons per sack of cement in the case of the air-entrained mixes containing Rock Island sand and gravel to 6.4 gallons for the three non-air-entrained mixes containing the crushed stone from Klamath Falls.

Two initial moist air curing periods at 70° F., of 28 and 180 days, were employed prior to starting the freezing and thawing tests. In the case of the 28-day curing period, the freezing tests were discontinued after 120 cycles. In the case of the 180-day curing period, they were continued through 270 cycles. All control specimens for strength determinations were stored continuously moist until tested at the same age as the companion specimens which were subjected to freezing and thawing.

Specimens for the freezing and thawing tests were 3- by 4- by 16-inch beams. They were frozen in water at approximately 0° F., followed by thawing in water at 70° F., using a 24-hour cycle. Stainless steel gage points were set in the ends of the specimens during molding so that measurements of length change could be made at periodic intervals.

Deterioration of the specimens was measured by the sonic test.[2] When the average of the three specimens showed a drop in sonic modulus of elasticity of approximately 40 percent, they were tested in flexure and one of each of the broken halves was tested in compression as a modified cube. At the same time the corresponding control specimens were tested in flexure and one of each of the broken halves was tested as a modified cube.

In the sonic test, changes in the elastic properties of the specimen are determined by measuring the change in its natural frequency of vibration. Assuming that the weights and dimensions of the specimens do not change during the test, the decrease in the square of this value, expressed as N^2, may be used in place of the computed modulus of elasticity to indicate deterioration. This common practice is followed in this report. The values for percentage reduction in N^2 are shown in tables 8 and 9.

As a convenience in studying relative durability as measured by the freezing and thawing test, Walker in 1944 proposed the use of a single value called for convenience the "durability factor."[3] This value is a function of the area under the curve which results when the dynamic modulus of elasticity N^2, expressed as a percentage of the value at zero cycles, is plotted in relation to the number of cycles. Wuerpel [4] has suggested a modification of Walker's formula based on the assumption that this curve is a straight line and that the degree of deterioration at a given end point of time or cycles is a more important factor than the shape of the curve.

The modification proposed by Wuerpel has been adopted by the American Society

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[2] *Application of sonic method to freezing and thawing studies of concrete*, by F. B. Hornibrook. American Society for Testing Materials, Bulletin No. 101, December 1939, p. 5.

[3] *Freezing and thawing tests of concrete made with different aggregates*, by Stanton Walker. Journal of the American Concrete Institute, June 1944, p. 573.

[4] *Discussion* by C. E. Wuerpel of Walker's paper (see footnote 3). Journal of the American Concrete Institute, November 1944, p. 580.

Table 9.—*Decrease in resistance of concrete to freezing and thawing, after initial moist curing of 180 days, as measured by decrease in* N^2

Aggregate combination and cement	Percentage decrease in N^2 after—cycles of freezing and thawing[1]														Durability factor[2]	
	20	40	60	80	100	120	140	160	180	200	220	240	250	270	At 120 cycles	At 270 cycles
NON-AIR-ENTRAINED CONCRETE																
I-1	4	8	9	10	15	19	26	31	37	[3]40	----	----	----	----	81	43
I-2	4	7	8	12	26	28	31	35	39	[3]40	----	----	----	----	72	43
I-3	4	13	15	19	24	27	30	34	35	35	37	37	[4]49	----	73	54
II-1	4	5	4	5	6	6	6	7	5	6	6	5	6	7	94	93
II-2	4	6	4	6	6	6	6	8	6	7	6	5	7	7	94	93
II-3	4	5	4	5	5	5	6	6	6	5	5	4	5	5	95	95
III-1	3	5	4	4	6	5	6	7	6	8	7	6	7	8	95	92
III-2	2	6	4	4	6	6	6	8	6	6	8	7	8	8	94	92
III-3	2	5	4	4	6	6	6	7	7	8	7	7	7	8	94	92
IV-1	12	18	21	27	34	[5]42	----	----	----	----	----	----	----	----	58	26
IV-2	14	20	20	21	25	26	32	35	36	[3]40	----	----	----	----	74	43
IV-3	16	21	24	29	33	35	[6]40	----	----	----	----	----	----	----	65	33
IV-4	16	20	24	28	32	38	[7]40	----	----	----	----	----	----	----	62	29
IV-5	10	12	12	14	16	19	23	28	28	32	37	39	[4]40	----	81	57
V-3	4	6	4	5	6	5	6	6	5	9	10	8	9	12	95	88
V-4	8	10	10	13	15	18	20	24	26	27	28	29	30	35	82	65
V-5	9	12	12	14	18	20	24	26	30	32	[8]40	----	----	----	80	49
AIR-ENTRAINED CONCRETE																
I-1	4	4	5	6	8	6	8	8	6	6	20	19	16	22	94	78
I-2	6	7	8	9	12	12	13	14	12	12	12	14	13	16	88	84
I-3	6	5	6	5	7	7	7	8	6	7	6	9	7	10	93	90
II-1	5	5	8	8	9	8	8	9	10	10	9	12	10	13	92	87
II-2	6	6	8	9	11	10	10	11	11	17	18	20	18	29	90	71
II-3	6	5	6	5	7	5	6	6	6	5	6	8	6	8	95	92
III-1	4	4	4	4	6	6	6	7	8	8	6	8	8	11	94	89
III-2	4	4	4	3	5	3	4	5	5	3	10	14	14	19	97	81
III-3	4	3	4	4	6	4	6	7	4	5	5	7	6	8	96	92
IV-1	10	11	11	11	12	12	14	13	14	13	14	16	16	22	88	78
IV-2	12	13	12	13	12	12	14	13	12	11	13	14	13	17	88	83
IV-3	8	8	8	9	10	9	9	8	8	7	8	10	9	13	91	87
IV-4	11	10	10	10	12	14	14	15	14	14	14	16	15	20	86	80
IV-5	10	10	10	9	10	11	10	12	11	10	10	13	12	16	89	84
V-3	6	7	6	5	8	7	8	7	7	6	6	8	8	9	93	91
V-4	9	10	8	12	14	14	16	15	16	15	16	20	19	23	86	77
V-5	9	10	19	18	21	21	20	23	20	22	25	27	27	30	79	70

[1] Each value is the average of tests on three beams except for I-1 and II-1 in the air-entrained group, for which two beams were tested.
[2] Method of determination similar to that described in Tests for Air-Entraining Admixtures for Concrete. A.S.T.M. designation C 233-49T.
[3] Final reading at 192 cycles.
[4] Final reading at 256 cycles.
[5] Final reading at 123 cycles.
[6] Final reading at 147 cycles.
[7] Final reading at 131 cycles.
[8] Final reading at 220 cycles.

for Testing Materials.[5] The durability factors employed in this paper and used in plotting figures 1–5 were computed from the formula given below, which is similar to that given in the A.S.T.M. publication referred to (the only difference is in the selection of 60 percent of the relative dynamic E as the end point for the freezing and thawing test):

$$DF=\frac{PN}{M}$$

where:

DF=durability factor of the concrete.

P=relative dynamic modulus of elasticity N^2 in percentage of the dynamic modulus of elasticity at zero cycles (values of P will be 60 or greater).

M=number of cycles at which durability factor is calculated.

N=number of cycles at which P equals 60 percent: If P has not reached 60 percent at the number of cycles M at which the durability factor is calculated, then $N=M$ and $DF=P$.

For the purpose of this discussion it is assumed that a durability factor DF of 75 represents the line of demarcation between durable and nondurable concrete. The durability factors for the various aggregate combinations and cements are shown in bar diagram form in figures 1–5. In each case, values are shown for both the non-air-entrained and the air-entrained concrete at the end of 120 cycles for specimens cured 28 days and at the end of 120 and 270 cycles for specimens cured 180 days.

Effects of Freezing and Thawing

Values for aggregate combinations I, the Mt. Hebron sand and Klamath Falls coarse aggregate, with cements 1, 2, and 3 are shown in figure 1. It will be noted that with all three cements, the non-air-entrained concrete containing this combination had very low resistance to freezing and thawing after 28 days of curing. Continuous moist curing to 180 days markedly improved resistance with all cements. Even with the additional curing, however, the durability factors at 120 cycles exceeded 75 in only one case (cement 1). When the 180-day cured specimens were subjected to 270 cycles of freezing and thawing, the durability factors were considerably reduced as compared to the values at 120 cycles, a further indication of the lack of durability of this aggregate combination.

The fact that the long curing period was of definite assistance in increasing durability would indicate the absence of any noticeable reaction between the aggregate and the alkalies in the cements in spite of the fact that the results of mortar-bar expansion tests with Klamath Falls basalt and cements 1 and 2 would indicate the possibility of a mild reaction (see table 4). The limited quantities of the aggregates available were insufficient for making tests to determine the contributing effect of the Mt. Hebron and the Klamath Falls materials separately to the poor freezing and thawing resistance encountered. Variations in the alkali content of the cements used with this aggregate combination did not appear to have much effect on resistance to freezing and thawing. Possibly any effect was masked by the low inherent resistance of the aggregate. These tests indicate that the combination of the Mt. Hebron sand and Klamath Falls coarse aggregate in non-air-entrained concrete is unsatisfactory for use in concrete for outdoor exposure where alternate freezing and thawing may occur.

The air-entrained concrete containing this aggregate combination showed, in all cases, greatly improved resistance as compared to the corresponding non-air-entrained concrete. However, the relative increases were much greater for the specimens cured 28 days than when the curing was extended to 180 days. Extending the freezing and thawing of the 180-day cured specimens to 270 cycles decreased the resistance in all cases, although the relative decreases were not nearly so great as for the corresponding concretes without entrained air. The durability factors of all of the air-entrained concrete specimens containing this aggregate combination exceeded 75 in all cases except one (with cement 2, cured 28 days).

From figure 2 it can be seen that aggregate combination II, the Umatilla sand and gravel, with and without air entrainment gave durability factors greater than 75, with two exceptions. These were the combinations with cement 2 in air-entrained concrete, cured 180 days and exposed to 270 cycles, and cement 3 without air entrainment, cured 28 days and exposed to 120 cycles. In both cases the durability factor was in excess of 60. These tests would indicate that the Umatilla sand and gravel are high-grade aggregates suitable for making concrete exposed to severe climatic conditions.

Durability factors for aggregate combination III, the sand and gravel from the Willamette River, are shown in figure 3. The similarity between figures 2 and 3 is immediately apparent, indicating that there is no practical difference in the concrete making properties of the Umatilla and Willamette aggregates, at least insofar as durability is concerned. It is interesting to note, when comparing figures 2 and 3, that in the case of the non-air-entrained

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[5] American Society for Testing Materials, Designation C 233-49T, Book of A.S.T.M. Standards, Part 3, 1949, page 821.

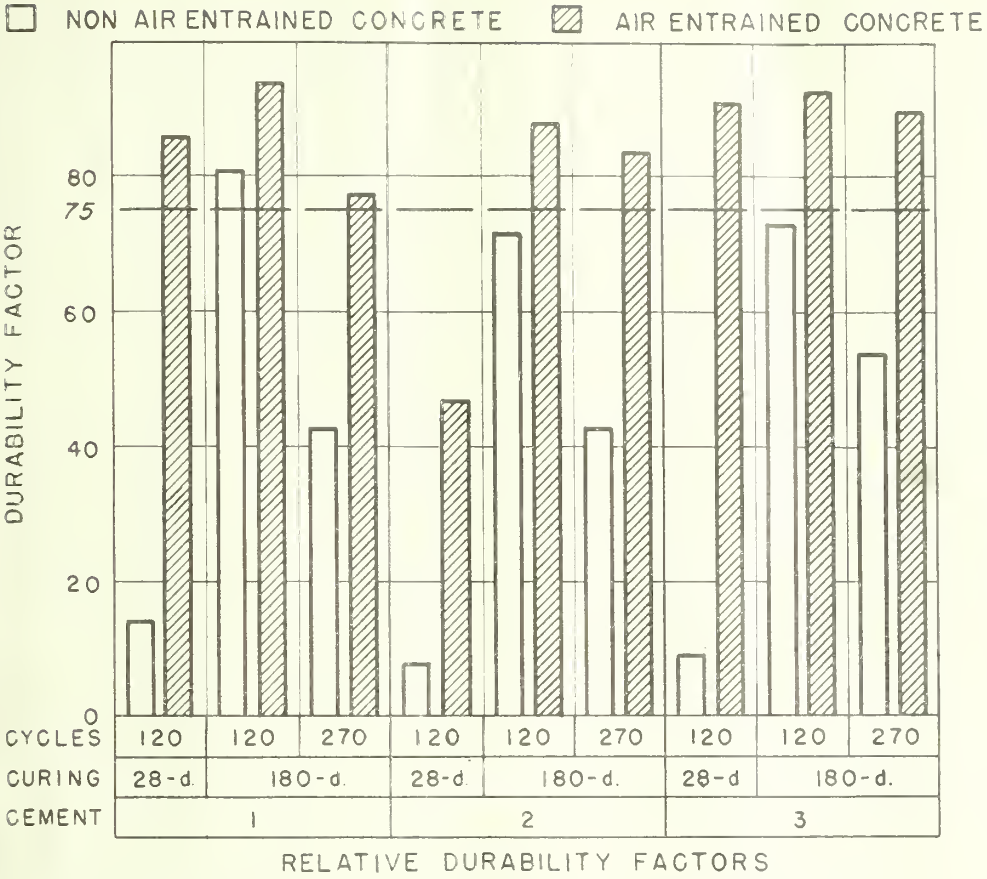

Figure 1.—Durability of aggregate combination I with cements 1, 2, and 3.

concrete, cured 28 days, the order of resistance as affected by the cement is the same—concrete containing cement 1 being the most resistant and concrete containing cement 3 the least resistant. However, this relation does not hold for the longer curing period and for this reason it may not be significant, insofar as ultimate durability is concerned.

As will be noted from figure 4, non-air-entrained concrete made with aggregate combination IV, Potomac River sand and gravel, developed poor resistance to freezing and thawing with all five cements. This aggregate gives good service in the relatively mild climate in the Washington, D. C., area, but these tests indicate that without air entrainment it would not be a suitable aggregate where freezing and thawing is severe. Potomac River aggregate has a wide range in mineral composition and it is possible that occasionally it may contain some reactive particles, but there is little or no direct evidence of reactivity. The comparatively poor resistance of concrete made with this aggregate is probably due to a combination of low bond strength and certain structural weaknesses of the aggregate particles. The resistance of the non-air-entrained concrete at 120 cycles was greatly improved by extending the curing period to 180 days. Resistance was also greatly benefited when air-entrainment was used, more so than was true for the concretes made with other aggregates, confirming the opinion that there is little possibility of reactivity even with high alkali cements.

It is of interest to note that increasing the curing period for air-entrained concrete from 28 to 180 days decreased the resistance slightly with all five cements. This suggests the possibility that, after long moist curing periods, some of the minute voids created by air entrainment may eventually fill with water. With minor exceptions, this same trend was noted also in all of the other aggregate combinations except combination I.

Aggregate combination V, the Rock Island sand and gravel, was from the eastern section of the State of Washington and is the same as the aggregate used in the tests by Tremper to which reference has been made. He found that this aggregate in concrete, when used with cements high in alkali and given prolonged moist curing prior to testing, broke down quite rapidly in freezing and thawing. Similar concretes containing the same aggregate but cements of lower alkali content gave fair resistance to the action of freezing and thawing.

As may be seen from figure 5, the Rock Island aggregates in combination with cement 5, having a total alkali content of 0.66 percent (expressed as sodium oxide), showed somewhat lower resistance at the end of 180 days of moist storage than was obtained with cement 4 (0.44 percent al-

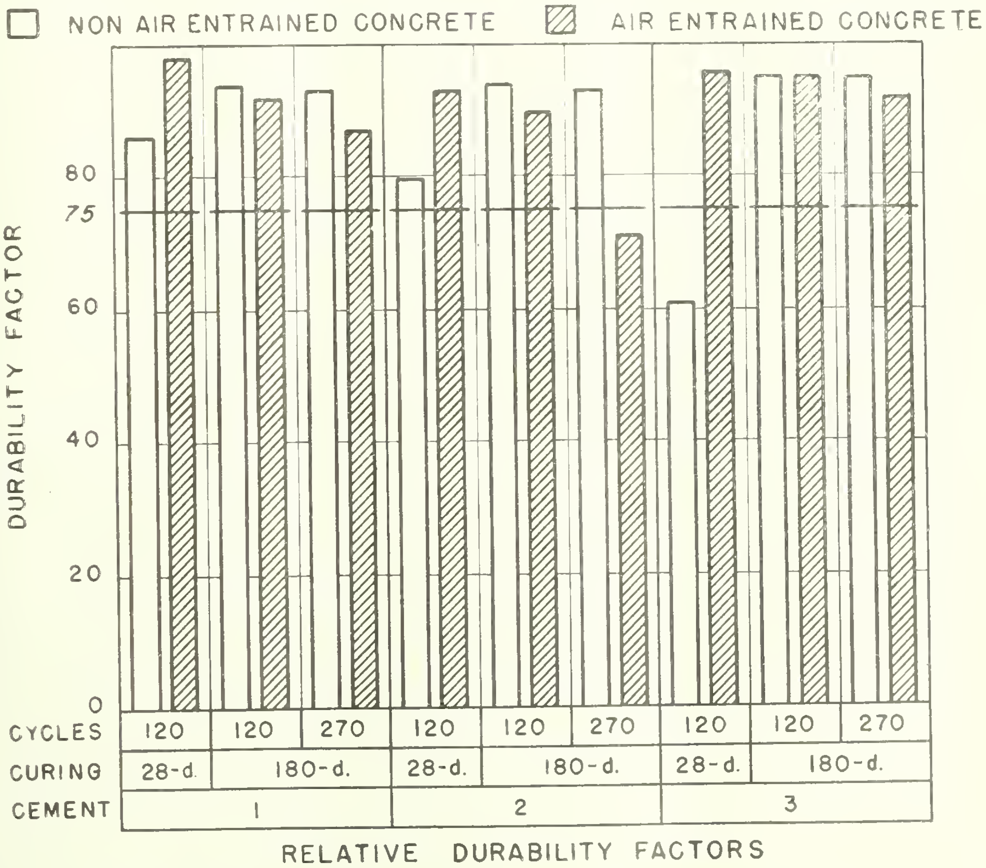

Figure 2.—Durability of aggregate combination II with cements 1. 2. and 3.

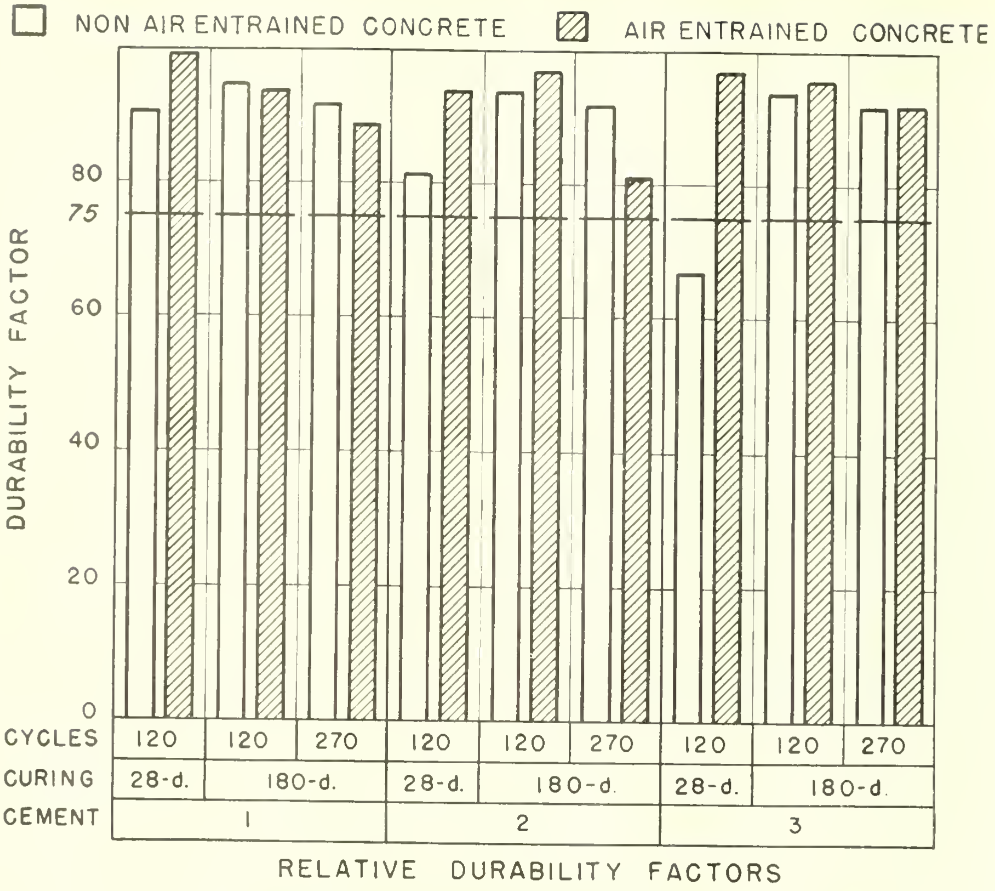

Figure 3.—Durability of aggregate combination III with cements 1, 2, and 3.

kali), and substantially lower resistance than was obtained with cement 3 (0.18 percent alkali). This is particularly noticeable at the end of 270 cycles of freezing and thawing. On the other hand, the results at the end of 28 days of moist storage were about the same for cement 5 (high alkali) as for cement 3 (low alkali). These results would seem to verify Tremper's conclusion that the Rock Island aggregate is slowly reactive with alkali in the cement. The non-air-entrained Rock Island concretes with cement 5 are the only ones which after 180 days of moist curing were less resistant than similar concrete cured 28 days. This would also seem to verify Tremper's conclusion. Except for this tendency to be slowly reactive, the Rock Island aggregate in combination with both high and low alkali cements had fairly good resistance to alternate freezing and thawing.

In the field where reactive aggregates were suspected, there has been evidence of map cracking of the concrete pavements. Several observers have thought that these cracks permitted easy entrance of water, hence making the concrete more susceptible to freezing and thawing action. This deductive reasoning may be correct, but it is difficult to simulate this condition in the laboratory. Careful observation of the test specimens failed to disclose evidence of cracking, but this in itself is not significant since small specimens of the size under consideration permit adjustment of the stresses

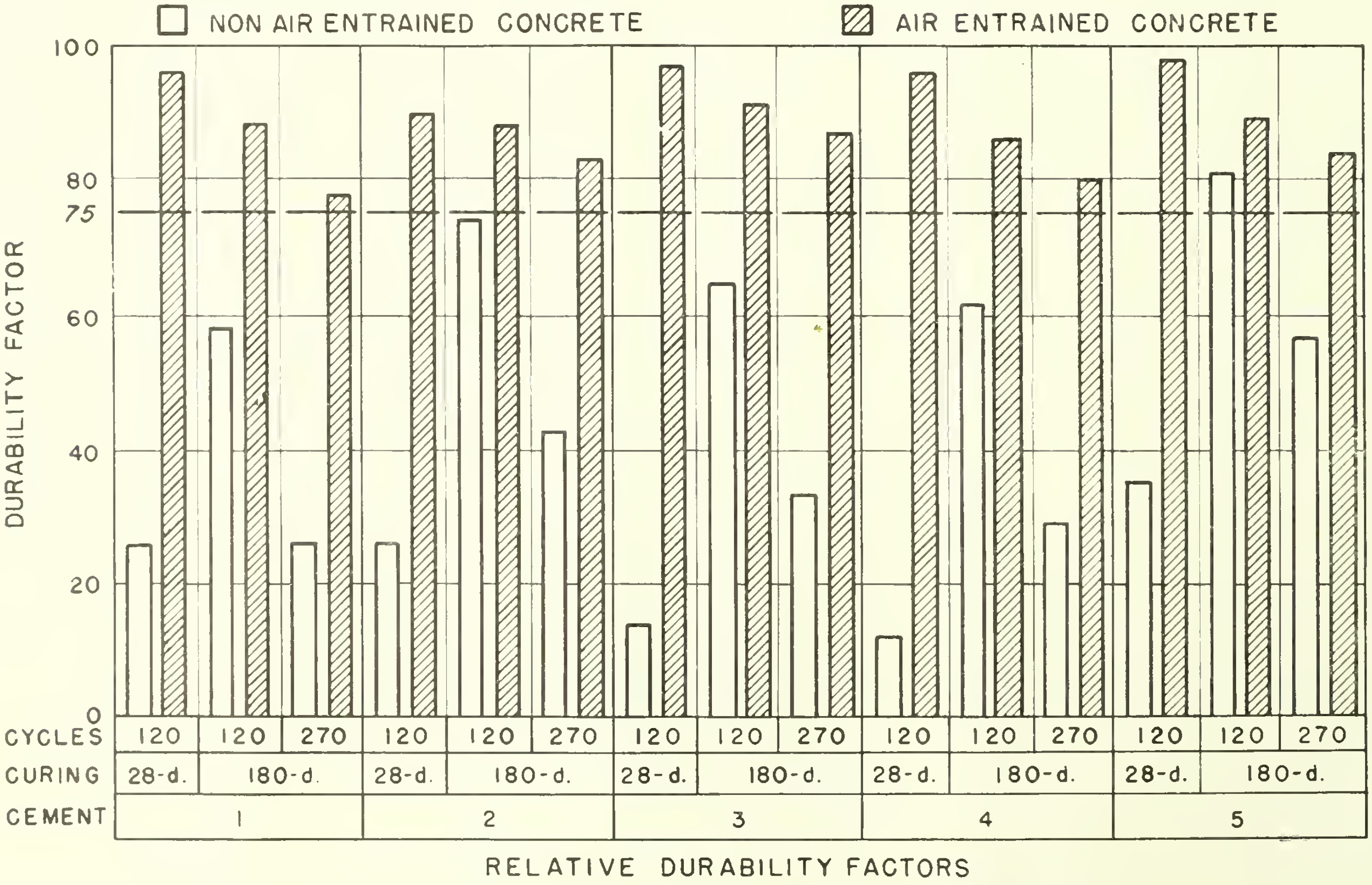

Figure 4.—Durability of aggregate combination IV with cements 1, 2, 3, 4, and 5.

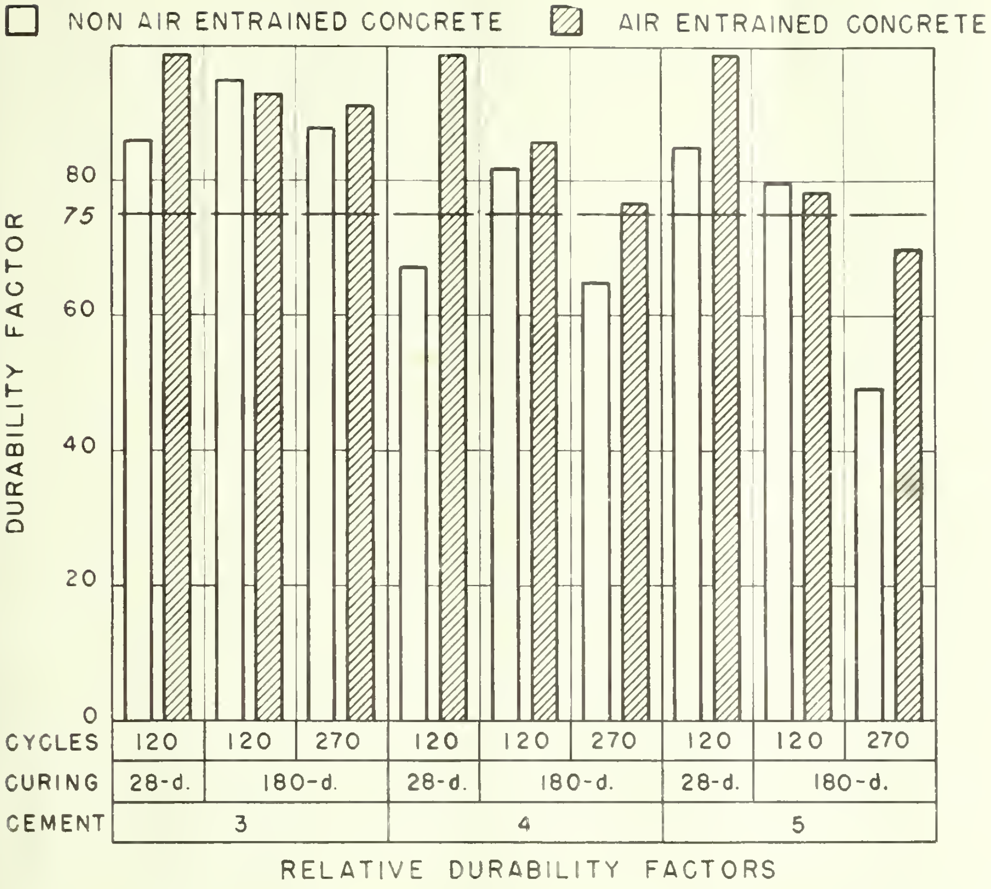

Figure 5.—Durability of aggregate combination V with cements 3, 4, and 5.

so that cracking occurs only under extreme conditions of volume change.

Length change measurements, shown in tables 10 and 11, were made on all the concretes subjected to freezing and thawing, but no significant information was developed that was not brought out by study of the durability factor.

Strength Tests

Tables 12 and 13 show the flexural strengths and the compressive strengths of modified cubes from the beams remaining after the freezing and thawing test was complete, as well as from control beams. It is customary to make such tests to confirm the results indicated by the drop in sonic *E*. For the non-air-entrained concrete, however, no direct comparisons can be made of the strengths of the concrete made of the various combinations of aggregates and cements, because of the variable number of cycles required to produce a 40-percent drop in sonic *E* and the consequent difference in age at time of test, and also because of the added curing obtained by the control specimens in moist air at 70° F. while the freezing tests were in progress. The frozen specimens do, of course, obtain some additional curing during the thawing phase of the cycle, but these temperatures are not favorable for promoting the curing action. This difference in curing is brought out by the observation that even though some of the frozen air-entrained concrete specimens showed little or no drop in N^2, a reduction of 10 to 15 percent in compressive strength of these same specimens was noted with an even greater reduction in the case of flexural strength.

Comparison of Durability Factors

To compensate for the variable number of cycles at which some of the specimens were tested, compressive and flexural strength durability factors somewhat similar to the durability factor calculated from sonic *E* were evolved. These factors were calculated by taking the ratio of the strength after freezing and thawing to the corresponding strength of the control specimens, multiplying this ratio by the number of cycles at time of test, and dividing the product by 120 in the case of the 28-day cured specimens or by 270 in the case of the 180-day cured specimens.

In table 14 the durability factors based on sonic determinations, on compressive strength loss, and on flexural strength loss have all been grouped for purposes of comparison. The durability factors for strength of specimens which withstood the maximum number of cycles are, of course, the same numerical values as the strength ratios shown in tables 10 and 11.

A comparison of the three durability factors indicates a reasonably good correlation as regards ability to differentiate between aggregate-cement combinations having poor resistance as measured by the sonic test and those combinations showing good resistance. For example, in all of the non-air-entrained concretes containing aggregate combinations I and IV, with all cements, the sonic test indicates low resistance (less than 75). For all of these combinations the corresponding factors calculated from the flexure test are also comparatively low (less than 50 in all cases). The corresponding compressive strength factors are also, with two exceptions, comparatively low (less than 70). It will be seen, therefore, that from the standpoint of selecting the definitely poor combinations any of the three factors may be used. The same general relations also hold for the three combinations which in general had good resistance as measured by sonic *E*. Six of the non-air-entrained combinations showed sonic *E* values of 75 or more at 120 cycles and seven combinations showed correspondingly high values at 270 cycles. For most of these the corresponding durability values in compression and flexure were also relatively high, although there were some individual exceptions to the trend as, for example, the comparatively low value for aggregate combination II with cement 2 in flexure at 120 cycles as compared to the sonic *E* and compression values.

It may be concluded that the reduction in sonic *E* reflects reasonably well the effects of freezing and thawing on both the compressive and flexural strength of the concrete, particularly in those cases where the resistance of the concrete is comparatively low.

It will be noted that the durability factor based on flexure strength is considerably lower than that based on compressive strength. Alternate freezing and thawing is probably more destructive to the outermost fibers than to the specimen as a whole. Since the flexural test is a measure of the bending strength of the outermost fibers it appears reasonable to expect lower relative strengths than indicated by the compression tests on the broken halves remaining after the flexure test.

(*Tables 10–14 are on the following pages.*)

Table 10.—*Decrease in resistance of concrete to freezing and thawing, after initial moist curing of 28 days, as measured by increase in length*[1]

Aggregate combination and cement	Percentage increase in length after—cycles of freezing and thawing (in thousandths of 1 percent)														
	8	16	24	32	40	48	56	64	72	80	88	96	104	112	120
Non-Air-Entrained Concrete															
I-1	8	23	42	----	----	----	----	----	----	----	----	----	----	----	----
I-2	14	42	59	----	----	----	----	----	----	----	----	----	----	----	----
I-3	17	39	48	----	----	----	----	----	----	----	----	----	----	----	----
II-1	5	6	6	7	9	10	11	13	16	17	19	18	20	20	24
II-2	5	6	5	9	10	12	15	15	16	18	24	24	25	26	27
II-3	3	5	3	7	11	13	17	21	28	32	36	38	39	41	48
III-1	4	4	5	6	6	7	8	12	13	14	16	17	17	16	20
III-2	1	5	6	7	9	10	12	15	17	17	21	24	22	22	24
III-3	3	4	2	6	10	12	19	24	31	33	37	39	41	44	46
IV-1	9	18	24	40	54	70	----	----	----	----	----	----	----	----	----
IV-2	9	14	20	30	38	49	----	----	----	----	----	----	----	----	----
IV-3	9	25	39	----	----	----	----	----	----	----	----	----	----	----	----
IV-4	13	37	58	----	----	----	----	----	----	----	----	----	----	----	----
IV-5	7	13	15	24	32	42	54	67	81	----	----	----	----	----	----
V-3	4	6	3	8	8	10	13	14	19	20	22	23	26	28	31
V-4	6	7	6	10	12	16	17	21	29	33	36	40	45	48	52
V-5	1	4	3	3	4	7	9	10	15	18	18	19	22	24	27
Air-Entrained Concrete															
I-1	4	4	6	7	8	9	11	13	15	15	15	16	17	18	18
I-2	2	3	8	7	9	12	15	18	23	22	23	25	27	29	31
I-3	3	5	6	9	8	10	12	14	16	17	18	18	20	20	22
II-1	4	2	4	4	4	4	7	6	11	10	9	10	11	11	11
II-2	1	1	3	3	4	6	7	7	10	10	9	10	9	11	11
II-3	2	5	3	4	4	6	9	9	9	11	10	12	11	11	10
[2]III-1	1	2	5	1	2	−1	4	8	9	8	8	8	9	11	11
III-2	1	2	5	3	4	3	7	7	10	9	7	9	8	9	9
III-3	1	3	4	1	4	4	7	5	9	7	7	8	7	9	8
IV-1	4	3	5	4	5	7	10	11	14	14	12	15	14	18	16
IV-2	3	5	7	6	9	7	11	11	15	16	16	18	18	18	17
IV-3	4	3	5	4	4	7	8	10	12	11	12	14	14	15	13
IV-4	3	3	5	5	5	8	9	11	14	13	12	12	14	15	16
IV-5	3	4	7	5	7	8	10	11	13	12	13	13	14	17	17
V-3	2	2	5	2	4	6	6	7	9	9	8	8	10	10	8
V-4	3	2	4	5	5	4	7	8	10	10	8	10	9	10	10
V-5	2	5	7	7	5	8	9	10	13	13	12	12	14	16	17

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[1] Each value is average of tests on three beams except as indicated.
[2] Average of two tests only.

Table 11.—*Decrease in resistance of concrete to freezing and thawing, after initial moist curing of 180 days, as measured by increase in length*[1]

Aggregate combination and cement	Percentage increase in length after — cycles of freezing and thawing (in thousandths of 1 percent)													
	20	40	60	80	100	120	140	160	180	200	220	240	250	270
Non-Air-Entrained Concrete														
I-1	3	4	6	8	12	19	24	33	42	----	----	----	----	----
I-2	3	4	5	6	14	17	20	24	30	----	----	----	----	----
I-3	2	4	4	6	9	11	12	16	18	19	21	23	25	----
II-1	2	3	4	3	6	7	8	9	10	11	15	14	12	15
II-2	2	2	1	2	3	4	4	7	7	7	8	8	9	11
II-3	2	3	4	2	4	5	5	7	8	8	10	10	11	14
III-1	1	2	2	1	4	4	4	6	8	7	10	10	11	12
III-2	2	3	2	4	4	4	5	6	6	7	8	7	7	9
[2]III-3	2	2	2	3	2	3	4	4	6	7	8	8	10	11
IV-1	9	15	23	29	41	53	----	----	----	----	----	----	----	----
IV-2	12	16	20	22	31	40	50	60	70	----	----	----	----	----
IV-3	12	21	28	38	50	62	77	----	----	----	----	----	----	----
IV-4	11	18	22	31	42	52	----	----	----	----	----	----	----	----
IV-5	9	13	16	22	29	47	45	57	68	76	88	101	105	----
V-3	2	3	2	4	6	5	6	9	10	12	13	15	17	19
V-4	4	7	10	12	16	20	24	31	32	38	46	50	53	61
V-5	8	10	14	18	22	28	33	38	45	48	59	65	----	----
Air-Entrained Concrete														
[2]I-1	0	0	2	2	2	1	2	5	5	2	9	8	8	6
I-2	0	1	2	3	3	3	6	8	5	6	7	8	8	7
I-3	0	0	0	2	0	1	4	5	2	0	3	2	5	5
[2]II-1	0	0	2	2	3	2	5	10	7	9	8	7	10	12
II-2	1	2	2	3	3	3	5	8	6	7	9	8	9	10
II-3	2	2	2	3	3	3	6	8	8	5	6	8	7	6
III-1	0	1	1	1	2	1	4	7	6	5	7	8	11	13
[2]III-2	0	1	0	1	0	−1	2	4	8	7	7	7	7	8
III-3	0	0	0	1	2	0	4	5	2	2	2	1	2	2
IV-1	5	8	10	14	14	16	20	26	25	25	29	31	34	38
IV-2	5	6	8	11	10	13	17	21	19	20	25	27	28	30
IV-3	2	1	2	4	5	7	9	12	12	11	15	15	16	18
IV-4	3	3	4	7	6	8	10	15	14	13	17	19	19	18
IV-5	6	7	10	13	14	17	22	27	28	30	32	36	38	40
V-3	2	1	1	3	2	3	6	8	7	7	8	9	9	11
V-4	2	3	4	6	6	8	10	14	13	13	16	18	19	20
V-5	4	5	5	10	12	12	14	20	21	20	26	29	30	38

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[1] Each value is average of tests on three beams except as indicated.
[2] Average of two tests only.

Table 12.—*Results of strength tests[1] after initial moist curing of 28 days and maximum of 120 cycles of freezing and thawing*

Aggregate combination and cement	Total age at test	Strength of control specimens		Average cycles of freezing and, thawing at test	Strength of specimens after freezing and thawing		Strength ratio[4]	
		Compressive[2]	Flexural[3]		Compressive[2]	Flexural[3]	Compressive	Flexural
Non-Air-Entrained Concrete								
	Days	*Lb./sq. in.*	*Lb./sq. in.*		*Lb./sq. in.*	*Lb./sq. in.*	*Percent*	*Percent*
I-1	58	5,900	725	24	4,790	395	81	54
I-2	55	5,830	775	21	5,030	350	86	45
I-3	55	5,540	725	21	4,810	330	87	46
II-1	187	7,730	950	120	6,390	620	83	65
II-2	187	7,910	965	120	7,170	410	91	42
II-3	187	7,790	1,005	120	6,540	515	84	51
III-1	187	7,250	970	120	6,020	590	83	61
III-2	187	7,380	910	120	6,120	640	83	70
III-3	187	7,570	955	120	5,640	475	75	50
IV-1	92	6,780	810	53	5,120	405	76	51
IV-2	92	7,080	800	53	5,440	[5]420	70	52
IV-3	67	6,620	820	32	5,300	355	80	43
IV-4	58	7,270	865	24	5,590	420	77	49
IV-5	118	6,720	860	75	5,120	350	76	41
V-3	187	7,950	925	120	6,280	620	79	67
V-4	187	8,190	1,050	120	6,730	415	82	40
V-5	187	7,180	910	120	6,140	495	86	54
Air-Entrained Concrete								
I-1	187	6,120	785	120	5,270	550	86	70
I-2	187	6,310	820	120	5,010	370	79	45
I-3	187	6,150	835	120	5,320	415	87	50
II-1	187	7,060	945	120	6,450	685	91	72
II-2	187	6,570	870	120	5,880	505	89	58
II-3	187	7,240	980	120	6,240	640	86	65
III-1	187	6,070	860	120	5,510	[6]750	91	87
III-2	187	6,460	880	120	5,500	675	85	77
III-3	187	6,540	900	120	5,900	720	90	80
IV-1	187	6,390	880	120	5,640	635	88	72
IV-2	187	6,330	815	120	5,430	580	86	71
IV-3	187	6,670	785	120	5,630	640	84	82
IV-4	187	6,200	860	120	5,870	655	95	80
IV-5	187	6,610	825	120	5,690	575	86	70
V-3	187	7,010	970	120	6,280	730	90	75
V-4	187	7,060	860	120	6,430	695	91	81
V-5	187	6,640	[5]965	120	6,040	650	91	67

[1] Each value is average of three tests except as indicated.

[2] Tested as modified cubes, specimens 3- by 3- by 4-inch depth (halves of flexure specimens remaining after tests).

[3] Specimens 3- by 4- by 16-inch, tested by center-point loading on a 12-inch span with the bottom (4-inch depth) as molded in tension.

[4] Values indicate percentage of strength of corresponding control specimen.

[5] Average of two tests.

[6] One test.

Table 13.—*Results of strength tests[1] after initial moist curing of 180 days and maximum of 270 cycles of freezing and thawing*

Aggregate combination and cement	Total age at test	Strength of control specimens		Average cycles of freezing and thawing at test	Strength of specimens after freezing and thawing		Strength ratio[4]	
		Compressive[2]	Flexural[3]		Compressive[2]	Flexural[3]	Compressive	Flexural
Non-Air-Entrained Concrete								
	Days	*Lb./sq. in.*	*Lb./sq. in.*		*Lb./sq. in.*	*Lb./sq. in.*	*Percent*	*Percent*
I-1	498	6,820	930	192	5,720	450	84	48
I-2	498	7,340	860	192	6,750	395	92	46
I-3	600	7,340	970	256	5,910	315	81	32
II-1	622	8,090	1,035	270	7,620	870	94	84
II-2	622	7,890	1,015	270	7,550	740	96	73
II-3	622	8,580	1,060	270	7,440	715	87	67
III-1	622	7,380	1,000	270	6,950	725	94	72
III-2	622	7,450	990	270	6,780	775	91	78
III-3	622	7,920	980	270	7,150	750	90	77
IV-1	400	7,050	865	123	5,750	395	82	46
IV-2	498	7,060	790	192	5,510	500	78	63
IV-3	434	8,130	900	147	5,190	445	64	49
IV-4	412	8,650	940	131	6,760	445	78	47
IV-5	600	6,850	885	256	5,620	395	82	45
V-3	622	8,320	915	270	7,520	660	90	72
V-4	622	8,340	1,010	270	6,470	440	78	44
V-5	572	7,030	820	220	5,940	360	84	44
Air-Entrained Concrete								
I-1	622	6,650	875	270	6,080	[5]520	91	59
I-2	622	7,200	920	270	6,070	600	84	65
I-3	622	7,090	945	270	6,200	690	87	73
II-1	622	7,840	1,050	270	6,740	[5]540	86	51
II-2	622	7,370	930	270	6,670	530	91	57
II-3	622	7,980	1,020	270	7,300	590	91	58
III-1	622	6,350	965	270	5,970	655	94	68
III-2	622	7,560	1,010	270	6,470	550	86	54
III-3	622	6,900	[5]1,060	270	5,840	700	85	66
IV-1	622	7,150	915	270	6,100	550	85	60
IV-2	622	7,240	850	270	6,150	570	85	67
IV-3	622	7,310	790	270	6,360	575	87	65
IV-4	622	6,960	905	270	6,560	730	94	81
IV-5	622	7,130	[5]895	270	6,010	570	84	64
V-3	622	7,500	990	270	6,860	710	91	72
V-4	622	7,900	925	270	6,730	560	85	61
V-5	622	7,350	880	270	5,840	455	79	52

[1] Each value is average of three tests except as indicated.

[2] Tested as modified cubes, specimens 3- by 3- by 4-inch depth (halves of flexure specimens remaining after tests).

[3] Specimens 3- by 4- by 16-inch, tested by center-point loading on a 12-inch span with the bottom (4-inch depth) as molded in tension.

[4] Values indicate percentage of strength of corresponding control specimen.

[5] Average of two tests.

Table 14.—*Comparison of durability factors*

Aggregate combination and cement	Durability factor of specimens after—					
	28-day curing and maximum of 120 cycles of freezing and thawing			180-day curing and maximum of 270 cycles of freezing and thawing		
	Sonic	Compressive	Flexural	Sonic	Compressive	Flexural
NON-AIR-ENTRAINED CONCRETE						
I-1	14	16	11	43	60	34
I-2	8	15	8	43	65	33
I-3	9	15	8	54	77	30
II-1	86	83	65	93	94	84
II-2	80	91	42	93	96	73
II-3	61	84	51	95	87	67
III-1	91	83	61	92	94	72
III-2	82	83	70	92	91	78
III-3	67	75	50	92	90	77
IV-1	26	34	23	26	37	21
IV-2	26	31	23	43	55	45
IV-3	14	21	11	33	35	27
IV-4	12	15	10	29	38	23
IV-5	35	48	26	57	78	43
V-3	86	79	67	88	90	72
V-4	67	82	40	65	78	44
V-5	85	86	54	49	69	36
AIR-ENTRAINED CONCRETE						
I-1	86	86	70	78	91	59
I-2	47	79	45	84	84	65
I-3	91	87	50	90	87	73
II-1	98	91	72	87	86	51
II-2	93	89	58	71	91	57
II-3	96	86	65	92	91	58
III-1	100	91	87	89	94	68
III-2	94	85	77	81	86	54
III-3	97	90	80	92	85	66
IV-1	96	88	72	78	85	60
IV-2	90	86	71	83	85	67
IV-3	97	84	82	87	87	65
IV-4	96	95	80	80	94	81
IV-5	98	86	70	84	84	64
V-3	99	99	75	91	91	72
V-4	99	91	81	77	85	61
V-5	99	91	67	70	79	52

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Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.
Title sheets for PUBLIC ROADS, volumes 24 and 25.

U.S. GOVERNMENT PRINTING OFFICE: 1951—951455

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF AUGUST 31, 1951

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$13,140	$16,968	$3,671	364.1	$6,384	$3,238	154.4	$17,497	$8,562	391.2	$40,849	$20,471	909.7
Arizona	671	4,471	3,181	99.1	2,822	1,983	42.7	3,797	2,694	39.3	11,090	7,858	181.1
Arkansas	2,993	10,813	5,781	388.2	6,889	3,441	208.1	13,668	6,971	360.9	31,370	16,193	957.2
California	6,981	24,413	4,100	60.3	12,915	5,611	137.9	67,070	32,225	238.7	104,398	41,936	436.9
Colorado	3,591	2,371	1,335	73.1	3,307	1,808	88.2	14,607	7,988	330.2	20,285	11,131	491.5
Connecticut	3,203	3,250	1,675	11.3	4,802	2,360	8.2	9,555	5,171	12.3	17,607	9,206	31.8
Delaware	1,541	4,532	1,613	37.4	458	227	9.2	3,749	1,810	17.6	8,739	3,650	64.2
Florida	4,061	12,324	6,291	321.9	6,098	3,099	85.0	21,370	10,657	403.0	39,792	20,047	809.9
Georgia	3,952	17,392	8,859	445.4	10,038	4,266	82.9	29,317	14,588	570.1	56,747	27,713	1,098.4
Idaho	4,019	9,729	6,106	328.0	1,379	848	54.7	8,460	4,778	252.9	19,568	11,732	635.6
Illinois	18,419	38,731	20,947	352.3	16,136	8,113	128.7	72,415	37,240	646.3	127,282	66,300	1,127.3
Indiana	11,562	32,764	16,414	174.3	10,233	4,965	134.8	17,529	8,922	164.9	60,526	30,301	474.0
Iowa	1,737	10,870	5,566	347.6	6,953	3,512	290.7	21,199	10,950	771.5	39,022	20,028	1,409.8
Kansas	6.563	8,442	4,064	1,239.6	3,732	1,871	223.7	14,321	7,142	529.2	26,495	13,077	1,992.5
Kentucky	667	20,502	10,734	375.6	7,292	3,704	153.9	14,232	7,043	189.8	42,026	21,481	724.3
Louisiana	2,401	17,781	8,539	80.8	9,880	4,658	76.8	20,397	10,336	207.9	48,058	23,533	365.5
Maine	1,936	8,261	4,500	69.3	2,087	1,001	20.2	7,434	3,872	62.8	17,782	9,373	152.3
Maryland	4,484	1,936	974	33.8	6,285	3,871	19.7	11,298	4,924	39.6	19,519	9,769	93.1
Massachusetts	439	14,804	6,547	32.9	6,275	2,793	2.0	70,124	34,911	59.1	91,203	44,251	94.0
Michigan	3,220	23,126	11,570	423.7	9,334	4,658	261.8	51,108	21,528	391.2	83,568	37,756	1,076.7
Minnesota	3,981	5,101	2,667	799.9	5,463	2,853	409.8	28,053	15,450	945.3	38,617	20,970	2,155.0
Mississippi	7,847	5,140	2,462	230.7	6,166	3,252	158.0	16,032	8,184	477.4	27,338	13,898	866.1
Missouri	8,472	23,780	12,424	764.5	11,846	5,899	246.7	38,125	20,080	682.1	73,751	38,403	1,693.3
Montana	4,688	15,047	9,208	450.9	4,000	2,290	82.4	13,376	8,082	205.8	32,423	19,580	739.1
Nebraska	8,202	10,838	5,717	406.8	6,181	3,099	136.9	17,938	9,318	562.7	34,957	18,134	1,106.4
Nevada	2,814	4,464	3,711	89.4	1,763	1,482	135.7	2,795	2,328	96.0	9,022	7,521	321.1
New Hampshire	2,603	2,244	1,306	13.9	637	295	5.3	5,476	2,712	48.4	8,357	4,313	67.6
New Jersey	2,834	12,949	6,355	25.9	9,421	4,698	14.9	16,860	7,993	17.7	39,230	19,046	58.5
New Mexico	1,833	4,349	2,783	172.2	1,689	1,076	52.7	10,997	7,027	190.2	17,035	10,886	415.1
New York	25,545	76,919	39,785	177.7	20,883	10,093	144.4	115,568	53,499	381.2	213,370	103,377	703.3
North Carolina	4,565	15,231	7,505	357.0	4,327	2,005	103.3	25,667	12,671	563.3	45,225	22,181	1,023.6
North Dakota	1,874	6,286	3,258	1,104.9	8,858	4,435	721.2	8,599	4,295	721.1	23,743	11,988	2,547.2
Ohio	12,588	33,695	15,972	256.9	3,368	1,753	63.5	78,807	39,800	387.4	115,870	57,525	707.8
Oklahoma	3,747	14,986	6,977	240.5	3,558	2,004	56.2	23,179	11,959	352.8	41,723	20,940	649.5
Oregon	752	1,971	1,159	19.1	2,639	1,420	43.3	17,610	9,837	229.6	22,220	12,416	292.0
Pennsylvania	11,572	24,953	12,469	79.5	9,646	4,821	34.0	67,837	33,910	194.2	102,436	51,200	307.7
Rhode Island	1,592	5,613	2,806	45.7	714	357	1.7	14,646	7,447	12.3	20,973	10,610	59.7
South Carolina	2,842	10,412	5,311	238.0	3,258	1,629	85.1	10,652	5,556	172.5	24,322	12,496	495.6
South Dakota	858	6,501	3,778	610.0	5,310	3,075	347.7	10,498	6,096	669.4	22,309	12,949	1,627.1
Tennessee	2,147	13,673	6,500	394.6	8,170	3,765	146.3	26,280	12,539	353.6	48,123	22,804	894.5
Texas	9,038	1,459	621	27.5	16,207	8,372	128.1	56,214	26,752	687.9	73,880	35,745	843.5
Utah	2,359	5,349	4,140	110.0	537	389	9.5	4,695	3,422	111.6	10,581	7,951	231.1
Vermont	637	5,269	2,844	59.4	367	185	6.5	4,673	2,323	26.3	10,309	5,352	92.2
Virginia	5,634	18,385	9,109	465.6	8,962	4,498	248.7	18,389	9,058	258.5	45,736	22,665	972.8
Washington	2,452	10,419	4,584	127.8	2,956	1,469	32.4	20,843	9,993	159.3	34,218	16,046	319.5
West Virginia	2,765	12,245	6,151	163.1	4,206	2,118	38.9	11,139	5,634	117.8	27,590	13,903	319.8
Wisconsin	6,401	19,950	10,994	391.1	7,840	3,742	179.8	20,921	10,331	460.9	48,711	25,067	1,031.8
Wyoming	305	2,154	1,494	48.2	1,715	1,095	60.0	7,657	4,978	222.8	11,526	7,567	331.0
Hawaii	1,081	8,445	3,692	14.4	1,683	829	12.0	9,356	3,504	23.7	19,484	8,025	50.1
District of Columbia	3,535	2,864	1,432	.7	200	100	.7	4,079	2,232	3.2	7,143	3,764	4.6
Puerto Rico	3,043	10,336	4,820	46.8	3,091	1,474	3.3	8,998	4,080	42.5	22,425	10,374	92.6
TOTAL	244,186	674,507	339,501	13,191.4	298,960	150,599	5,897.6	1,205,106	603,402	15,056.0	2,178,573	1,093,502	34,145.0

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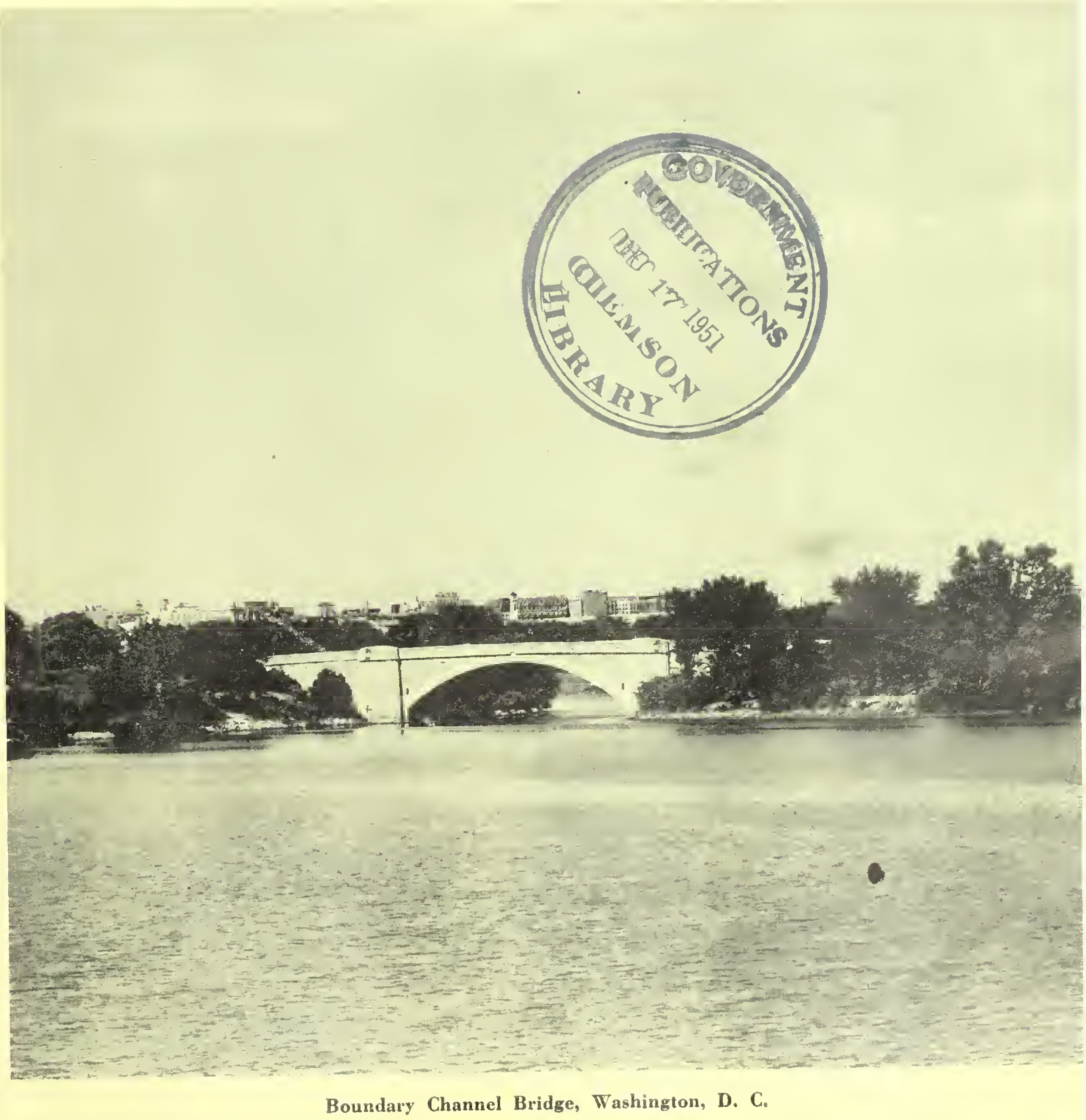

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BUREAU OF PUBLIC ROADS

U. S. DEPARTMENT OF COMMERCE

E. A. STROMBERG, Editor

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Observed Settlements of Highway Structures Due to Consolidation of Alluvial Clay

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by E. S. BARBER
Highway Engineer

Laboratory consolidation tests, coupled with analysis of field conditions indicated by subsurface explorations, enable the engineer to anticipate the amount and rate of settlement of embankments and bridges under increased loads. In this article observed settlements at four locations are correlated with calculated values obtained from laboratory test results. Actual settlements were found to be in substantial agreement with the calculated values, the maximum difference being about 20 percent. Primary consolidation accounted for most of the observed settlements.

LABORATORY consolidation tests plus analysis of field conditions indicated by subsurface explorations are useful in estimating the amount and rate of settlement of embankments or bridges to be expected due to an increase in applied load. However, correlation of such analysis with recorded field displacements is needed, particularly in evaluating boundary drainage conditions for primary consolidation and in evaluating secondary time-consolidation effects which are independent of boundary drainage conditions.

This paper presents observations of settlements at four different sites along the Potomac River near Washington, D. C., as shown in figure 1, and their correlation with laboratory test results and analysis. Field data were obtained by the Bureau of Public Roads and the District of Columbia Department of Highways, and the tests were made in the Bureau laboratory.

Summary

The total settlements indicated by the field observations were in substantial agreement with the values calculated from laboratory compressibilities. The maximum difference was about 20 percent. Primary consolidation accounted for substantially all of the observed settlements except for the peaty material at one location, where secondary consolidation was quite appreciable. The rate of consolidation in clay with sand lenses was somewhat more rapid than that calculated for purely vertical consolidation, although much less rapid than would have been derived for free draining lenses.

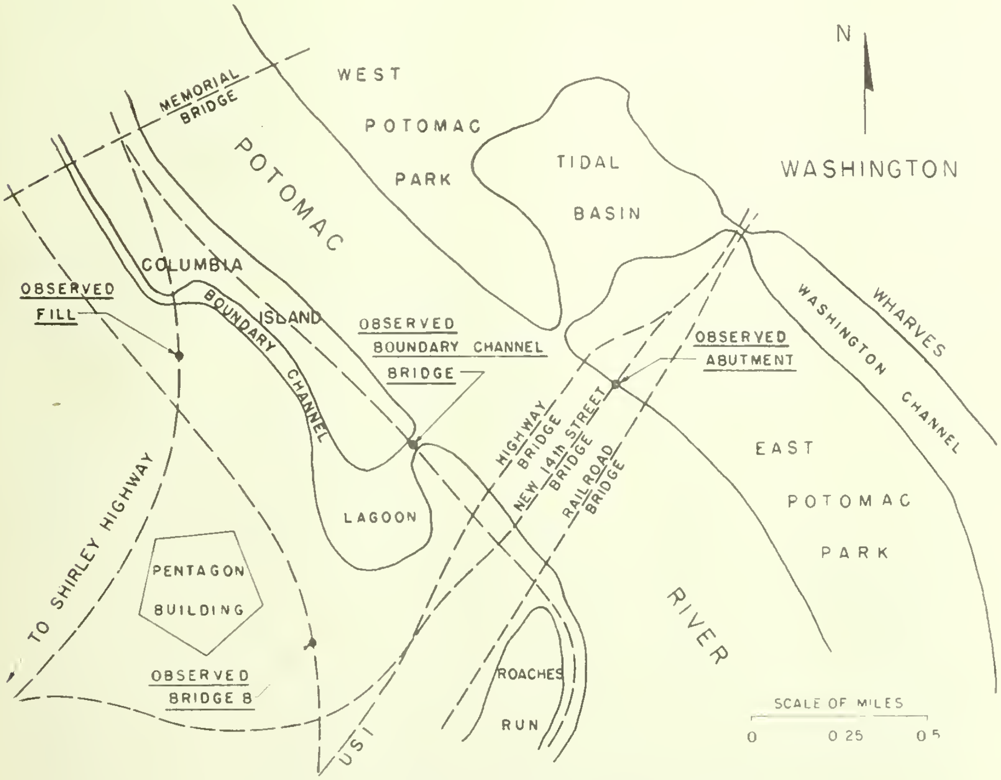

Figure 1.—Location of observed settlements.

Test Methods

Consolidation tests were made on undisturbed samples taken from each soil layer by the suggested method of test for consolidation of soil.[1] Illustrative consolidation test results are given in table 1 and the physical properties of the several soils over which settlements were observed are shown in table 2. Using the data from the consolidation tests, the coefficients of compressibility and consolidation for the loads appropriate to each problem were calculated by the methods shown in figure 2.

Loading intervals of 24 hours were used for obtaining all reduction in thickness values except for the samples from the upper layer of Bridge 8. In the latter instance, the time interval was 96 hours.

Fill on Three Compressible Layers

As part of the road network around the Pentagon, a 35-foot rolled fill of silty soil was constructed over a tidal flat at the location marked "observed fill" in figure 1.

Samples taken from borings at this location disclosed three layers of compressible soil, as shown in the cross section at the top of figure 3. Therefore, settlement of the embankment was anticipated but it was decided to raise the grade line of the roadway on the embankment at the bridge ends where necessary rather than excavate the soils in layers 1, 2, and 3 of the foundation and thus eliminate the settlement.

Using the coefficients of compressibility and consolidation obtained from consolidation tests, the computed time-settlement curves, shown in figure 3, were drawn before construction started. The points for

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[1] *Procedures for testing soils*, American Society for Testing Materials, 1950, p. 240.

Table 1.—*Illustrative consolidation test results* [1]

	Layer		
	1	2	3
Percentage reduction in thickness for pressures of:			
0.02 kip per sq. ft	0.0	0.0	0.0
1 kip per sq. ft.	5.1	14.8	1.5
2 kips per sq. ft	9.5	28.1	2.7
4 kips per sq. ft	18.4	39.7	4.3
8 kips per sq. ft	25.1	49.7	6.4
Average percentage of consolidation after:			
0.09 minute	10	9	22
.25 minute	14	13	28
.49 minute	18	17	33
1 minute	24	23	38
4 minutes	38	33	61
25 minutes	57	51	81
Initial sample thickness, inches	0.49	0.49	0.49
Initial moisture content, percent dry weight	73	232	22
Initial wet density, pounds per cu. ft	95	73	128

[1] The fill on the Pentagon network, designated "observed fill" in figure 1.

plotting the curves were calculated from the following formula based on an average vertical permeability and average compressibility:

$$t=T\sum mH\sum\frac{H}{mc_v} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which

t=time in years for a given degree of settlement.
T=time factor.
m=coefficient of compressibility.
c_v=coefficient of consolidation.
H=thickness of each layer.

The calculations for the two summations are shown in table 3. The time factors T are taken from table 4. In table 3, 2.69 feet is the maximum calculated settlement in the three layers due to the weight of the 35-foot fill. Thus, for 50-percent consolidation, or 1.34 feet, and drainage from two faces, the time would be, using equation (1), $t=0.05\times0.625\times6{,}130=191.6$ days$=0.52$ year.

Similar calculations were made for other percentages of consolidation to obtain data for plotting the computed curve for two drainage faces in figure 3. Adjustments were made for the period of load increase indicated at the top of the graphs in figure 3.

A similar procedure was followed in deriving the computed curve for one drainage face. For these computations the ratio of the pressure at the drainage face to presure at the impervious face was assumed as 1.00.

The observed settlement curve of figure 3 was drawn by plotting changes in elevation of the settlement plate. The settlement plate was placed during construction of the embankment and consisted of a steel plate 24 inches square to which was screwed a stem consisting of a 1-inch diameter pipe. The plate and first section of stem were placed 2 feet below the original ground surface and a 2-inch guard pipe was placed around the stem. Additional sections of stem and guard were added as the height of the fill increased. After completion of the fill, the guard pipe was capped. Elevation readings referred to a permanent bench mark were taken on the stem at regular time intervals and the fill settlement calculated.

A comparison in figure 3 of observed settlements with those calculated for two drainage faces indicates that the fill may have acted initially as a drain but that its resistance to flow of water from the foundation increased as it became saturated.

Consideration of degree of consolidation in each layer as affected by proximity to a drainage face [2] would make considerable difference in time calculations, but less difference than the uncertainty of boundary drainage. The section of curve designated in figure 3 as "secondary rate" will be discussed subsequently.

Calculations based on samples taken at two other locations on the same fill indicate ultimate settlements of 0.81 and 3.62 feet although the observed settlements were both approximately the same as shown in figure 3. This shows that the subsoil was more uniform with respect to support of a 35-foot fill than indicated by the samples obtained from the three individual borings.

Elevations taken on temporary stakes and the pavement at the top of the fill showed the same settlement as the plate below the fill, indicating that there was no consolidation within the fill. A similar record of no movement within a rolled fill was previously reported in PUBLIC ROADS.[3]

Displacements at the Boundary Channel Bridge

In 1931, as part of the Memorial Highway to Mount Vernon, a bridge was built over Boundary Channel connecting the river bank to Columbia Island, newly formed by hydraulic fill. The sketch at the top of figure 4 shows the deep layer of organic clay under the Boundary Channel Bridge and the adjacent fill. The bridge, consisting of twin cantilevers with a small suspended span, was supported on piles to adequate bearing and did not settle. However, the bridge buckled due to the lateral pressure transmitted from the adjacent fill placed on the clay. A bench mark was set in the fill on June 1, 1934. The time, measured from the mean time of placement of the fill almost 4 years previous, was plotted against the observed fill settlement as shown in figure 4. The primary consolidation relation for one-dimensional drainage, as given in table 4 for two drainage faces, was adjusted in scale to fit the plot of fill settlement as closely as possible, and the fitted theoretical curve, shown in figure 4, was found by successive trials.

A record of the fill settlement between June 24, 1932, and June 1, 1934, was subsequently found. This record, as shown in figure 4, agreed with the fitted theoretical curve indicating that the settlement was due to primary consolidation. This settlement due to consolidation is additional to any that took place due to lateral displacement at the time of placing the fill.

The discrepancy between the fitted theoretical curve and the actual fill settlement after 10 years is due to the 5-percent load increase caused by the addition of 2 feet of fill material, which was necessary to maintain a satisfactory riding surface.

Table 5 shows the consolidation properties of samples of clay taken from borings

Table 2.—*Properties of alluvial clays*

	Pentagon fill			Old Boundary Channel Bridge, average	Bridge 8, Pentagon network		New 14th St. Bridge, lower layer
	Layer 1	Layer 2	Layer 3		Upper layer	Lower layer	
Percentage passing:							
No. 10 sieve	100	100	100	100	74	99	100
No. 40 sieve	99	94	98	99	72	97	87
No. 200 sieve	88	70	75	85	59	65	71
0.005 mm	47	25	45	30	27	22	32
Liquid limit	56	120	33	51	61	23	58
Plastic limit	18	24	16	13	13	6	26
Coefficient of consolidation, ft. sq. per day	0.14	0.24	0.10	0.28	0.17	0.46	0.04
Compressibility, sq. ft. per kip	0.043	0.090	0.0088	0.043	0.030	0.006	0.015

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[2] *Simultaneous consolidation of contiguous layers of unlike compressible soils*, by Hamilton Gray. Transactions of the American Society of Civil Engineers, 1945, p. 1327. Discussion, p. 1345.

[3] *Research on the construction of embankments*, by Henry Aaron, W. T. Spencer, and H. E. Marshall, PUBLIC ROADS, vol. 24, No. 1, July-Aug.-Sept. 1944.

made at the site of the Boundary Channel Bridge. The variations in the coefficients of consolidation for the samples indicate some sandy strata for which the continuity and extent could not be determined. Considering the pressure of 2.7 kips per square foot, due to the weight of granular fill on the 65-foot layer of organic clay, the 7.0-foot settlement (4.8+2.2) indicated in figure 4 would require a compressibility of 0 040 which compares well with the 0.043 average of the laboratory test results shown in table 5. Assuming vertical drainage only, the settlement record indicates a coefficient of consolidation c_v of 0.28 foot squared per day. This agrees with the average c_v shown in table 5. However, the weighted average vertical c_v is calculated as follows:

$$c_v = \frac{1}{\text{avg. } m \times \text{avg. } \frac{1}{c_v m}} = \frac{1}{0.043 \times 229} =$$

0.10 foot squared per day.

This value is so low as to indicate some lateral drainage which could not be evaluated from the data available before the recording of field settlements.

The similarity of the curves for pier rotation and fill settlement in figure 4 suggests that the lateral movement of the piers toward each other is controlled by the lateral consolidation of the clay between the pile groups. Struts placed between the piers below water in August 1945 have had no apparent effect on the rotation of the piers.

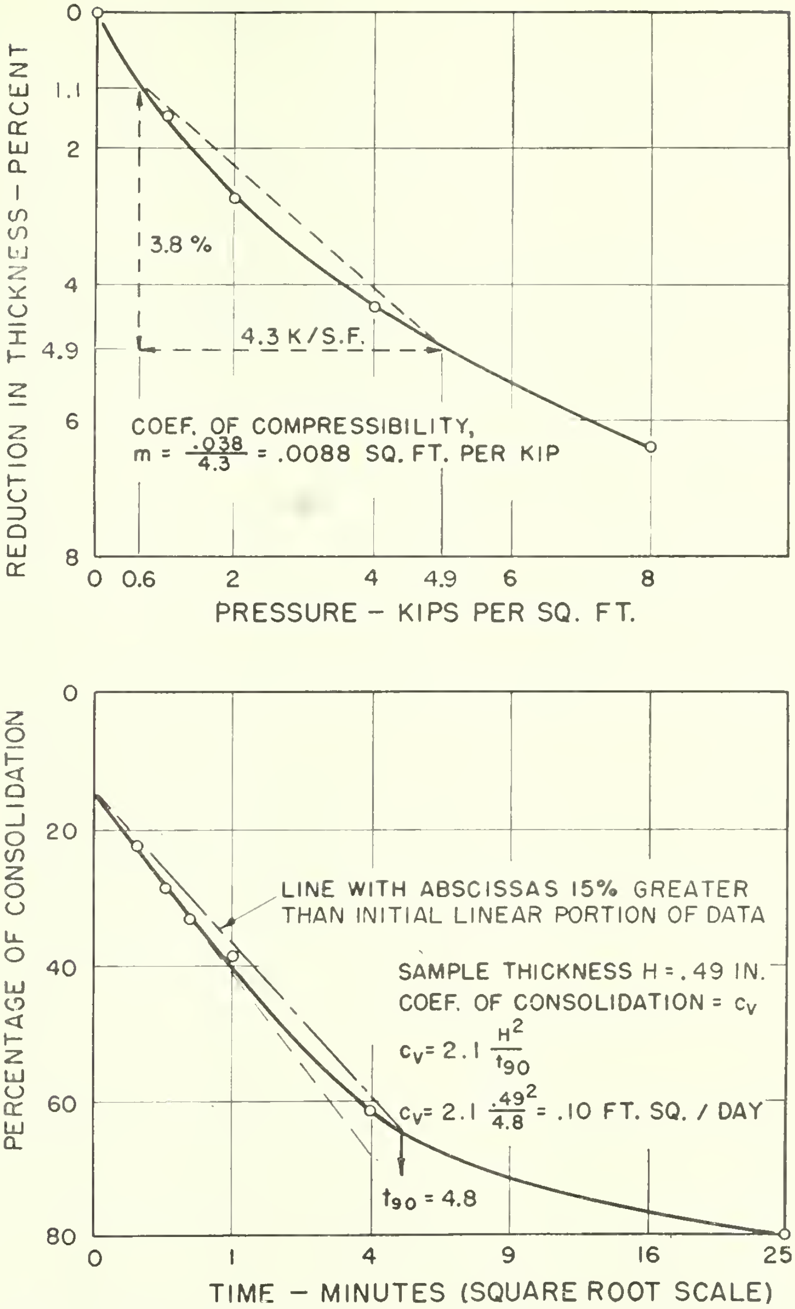

Figure 2.—Plot of consolidation test results: third layer of Pentagon fill.

Secondary Consolidation

The foregoing calculations have assumed primary consolidation based on soil permeability and location of drainage boundaries. Laboratory time-consolidation records often indicate that primary consolidation is followed by a secondary consolidation characterized by an approximately linear relation between thickness change and the logarithm of time. The time for secondary consolidation is assumed to be independent of the location of drainage boundaries and appears to be unimportant until the primary consolidation has slowed down so that its rate is equal to the secondary rate, whereupon the secondary rate controls.

Predicted rate of secondary consolidation, based on a projected linear relation between time from 1 to 24 hours and thickness change of samples in the laboratory, is shown between 7 and 9 years in figure 3, and between 16 and 20 years in figure 4. The fact that the rate of observed movement is considerably greater than the secondary rate indicates that primary consolidation is still predominant.

Evidence of more important secondary consolidation was found at Bridge 8, a grade separation on the Pentagon road network, in a silty clay layer, which was peaty in the upper portion as indicated by the profile in figure 5. To support wing walls at elevation 25, piles were driven through 15 feet of rolled fill and 10 feet of dump fill into the clay. Due to the resistance to driving built up in the fill, the piles did not reach the sand and gravel below the clay. When fill was placed around the walls, settlements were observed as shown in figure 5.

In analyzing the record, the observed settlement values were adjusted to eliminate the settlement due to the October 1942 fill, leaving primarily the settlement due to the August 1942 fill. A curve for primary consolidation for simple vertical drainage was fitted to the adjusted curve. As shown in figure 5, the fit was very good up to 8 months or 90 percent of the indicated primary consolidation.

The thickness change of the laboratory samples of the peaty clay plotted against logarithm of time was linear from 1 to 96 hours and showed a secondary settlement per logarithmic cycle of 20 percent of the total for each load increment. If this secondary consolidation is assumed to start at 8 months, it would account for an additional settlement at 80 months of 20 percent of the indicated primary settlement or 0.2×0.71=0.14 foot. The observed difference between the adjusted observation and the fitted primary consolidation at 80 months is 0.96−0.71 or 0.25 foot. The excess (0.25−0.14=0.11) may be due to the secondary consolidation from the fill placed in January 1942. It should be noted that the record of observed settlement is concave upward, indicating that the linear relation shown up to 4 days in the laboratory is not maintained up to 80 months.

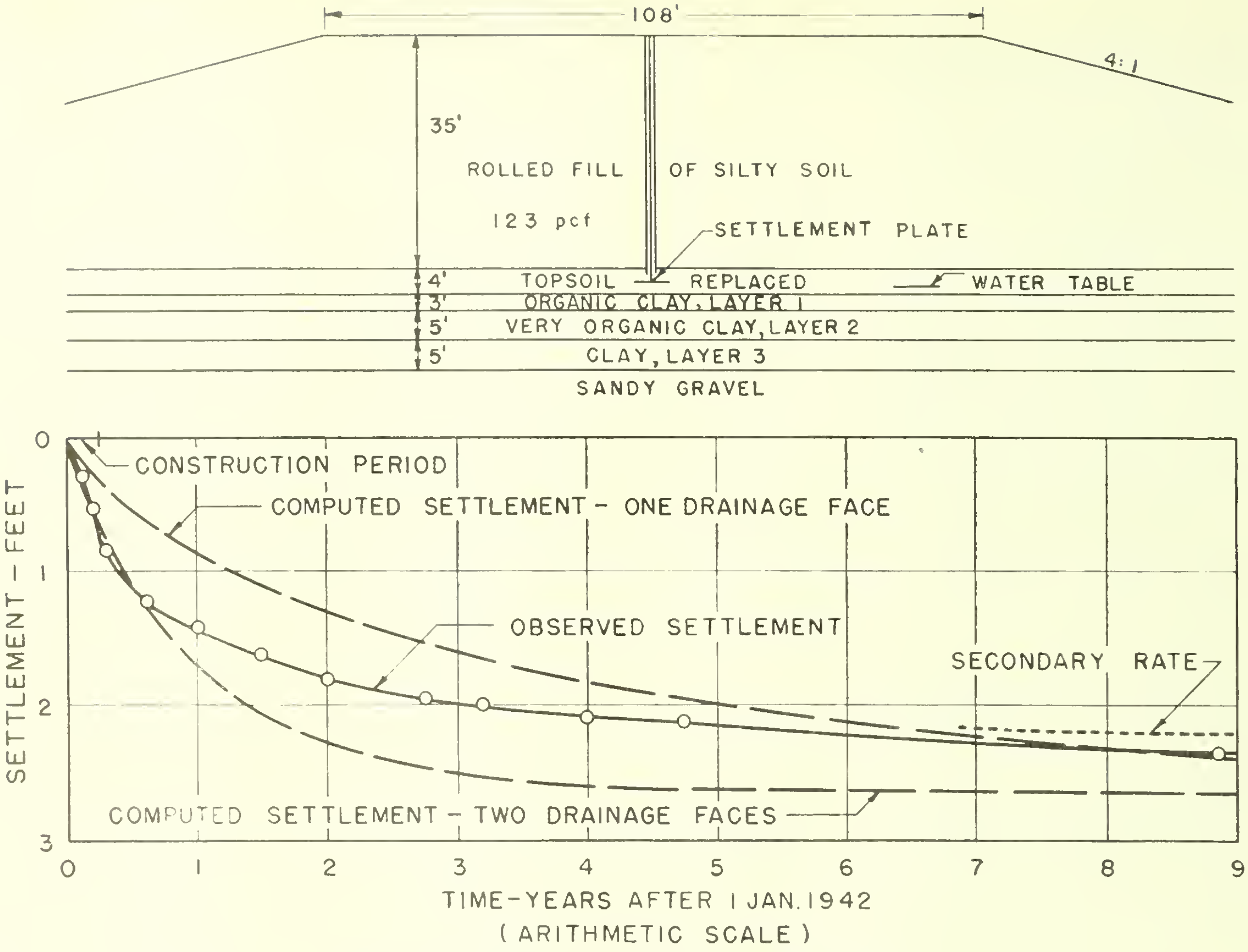

Figure 3.—Settlement of fill on Pentagon network.

Abutment of New Fourteenth Street Bridge

The north abutment of the new Fourteenth Street Bridge over the Potomac River at Washington was supported on piles driven to good bearing according to pile-driving formulas and short-time loading tests. Despite the fact that borings showed soft organic clay below the piles, the design was approved because no trouble had been experienced with the old bridge, which is situated nearby on a similar foundation. Subsequent investigation disclosed that the old abutment had settled 11 inches but without damaging the simply supported truss span. The presence of the settlement was obscured by the general settlement of the adjacent reclaimed marsh and the use of the abutment as a bench mark. An equal settlement could not be tolerated on the new bridge with continuous plate-girder spans. When the new abutment had settled 18 inches at the fill end of the wing walls and 2 inches at the bridge seat, it was decided to underpin the structure with steel piles driven to sand and gravel below the soft clay. An important factor in making this decision was the fact that the bridge seat had also moved 3 inches toward the fill.

The abutment, as shown at the top of figure 6, was built above the original ground and the rolled fill placed, the middle of the filling period being in February 1949. Four months later, continuous observations of settlement were started at the bridge seat and at the opposite end of the wing wall. The fill and the wing wall settled together due to the compression of both the upper (elevation +7 to −40) and lower (elevation −40 to −80) compressible soil layers. This settlement is shown by the solid portion of the lower curve in figure 6. The settlement of the bridge seat was due primarily to the consolidation of the lower layer, to which the piling was driven; there was no fill directly above the area under the bridge seat. The solid portion of the upper curve in figure 6, obtained by plotting settlement of the bridge seat against the square root of time, is linear except for the rebound due to excavation for underpinning which started 10 months after construction.

Calculations from laboratory tests of samples taken from borings showed that the total settlement due to consolidation of the lower layer caused by the load from the fill and abutment would be 8 inches under the bridge seat and 14 under the wing-wall end.

The settlement in the upper soil-layer under the end of the wing wall was calculated by subtracting 14/8 of the observed bridge seat settlements from the observed wall settlements. The calculated settlement as related to time is shown as the solid portion of the center curve of figure 6. A theoretical primary curve for one dimensional consolidation was fitted to the calculated curve for the upper layer. The portions of the theoretical curve that extend be-

Table 3.—*Time-consolidation of three-layer system*

Layer	Thickness H	Coefficient of compressibility m	Coefficient of consolidation c_v	mH	$\frac{H}{mc_v}$	Settlement under 4.3 kips per sq. ft.[1] (4.3 mH)
	Feet	*Sq. ft./kip*	*Ft. sq./day*			*Feet*
1	3	0.0434	0.14	0.130	490	0.56
2	5	.0902	.24	.451	230	1.94
3	5	.0088	.105	.044	5,410	.19
Total	13	....	....	.625	6,130	2.69

[1] 4.3 kips per square foot is approximately the load applied to the three layers by the rolled fill, 35 feet high, with a density of 123 pounds per cubic foot.

yond the calculated values are shown by the dashed lines on the middle curve of figure 6.

When the underpinning was complete, the movement of the wing wall stopped but the fill continued to settle. By adding 14/8 of the projected bridge seat settlement to the fitted primary consolidation curve for the upper soil layer, a predicted curve for fill settlement was derived and is shown as the dashed extension of the lower curve in figure 6. A check observation made 24 months after construction and plotted in figure 6 shows excellent agreement between the computed and the observed fill settlement.

Based on the 8-inch settlement of the bridge seat calculated from test results on the lower layer, 25 percent of primary consolidation occurred in 6 months, indicating a coefficient of consolidation of 0.11 foot squared per day based on vertical consolidation. As shown in table 2, the average laboratory value is 0.04, showing that the sand lenses had appreciable effect in accelerating the settlement.

Table 4.—*Effect of boundaries on time-consolidation*

	Degree of consolidation								
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
TIME FACTOR $T=\frac{c_v t}{H^2}$									
Ratio of pressure at drainage face to pressure at impervious face:									
0	0.049	0.100	0.154	0.217	0.29	0.38	0.50	0.66	0.95
.2	.027	.073	.126	.186	.26	.35	.46	.63	.92
.4	.016	.056	.106	.164	.24	.33	.44	.60	.90
.6	.012	.042	.092	.148	.22	.31	.42	.58	.88
.8	.010	.036	.079	.134	.20	.29	.41	.57	.86
1.0	.008	.031	.071	.126	.20	.29	.40	.56	.85
1.5	.006	.024	.058	.107	.17	.26	.38	.54	.83
2	.005	.019	.050	.095	.16	.24	.36	.52	.81
3	.004	.016	.041	.082	.14	.22	.34	.50	.79
5	.003	.013	.034	.069	.12	.20	.32	.48	.77
10	.003	.011	.028	.060	.11	.18	.30	.46	.75
Infinity	.002	.009	.024	.048	.09	.16	.28	.44	.73
Two drainage faces	.002	.008	.018	.031	.05	.07	.10	.14	.21
TIME FACTOR $T=\frac{c_v t}{D^2}$									
Ratio of well diameter to effective spacing *D:*									
0.01	.046	.104	.167	.24	.33	.44	.58	.78	1.10
.025	.032	.075	.124	.180	.25	.33	.44	.58	.86
.1	.014	.037	.064	.096	.132	.178	.24	.32	.44
.2	.006	.019	.035	.054	.077	.105	.14	.19	.29

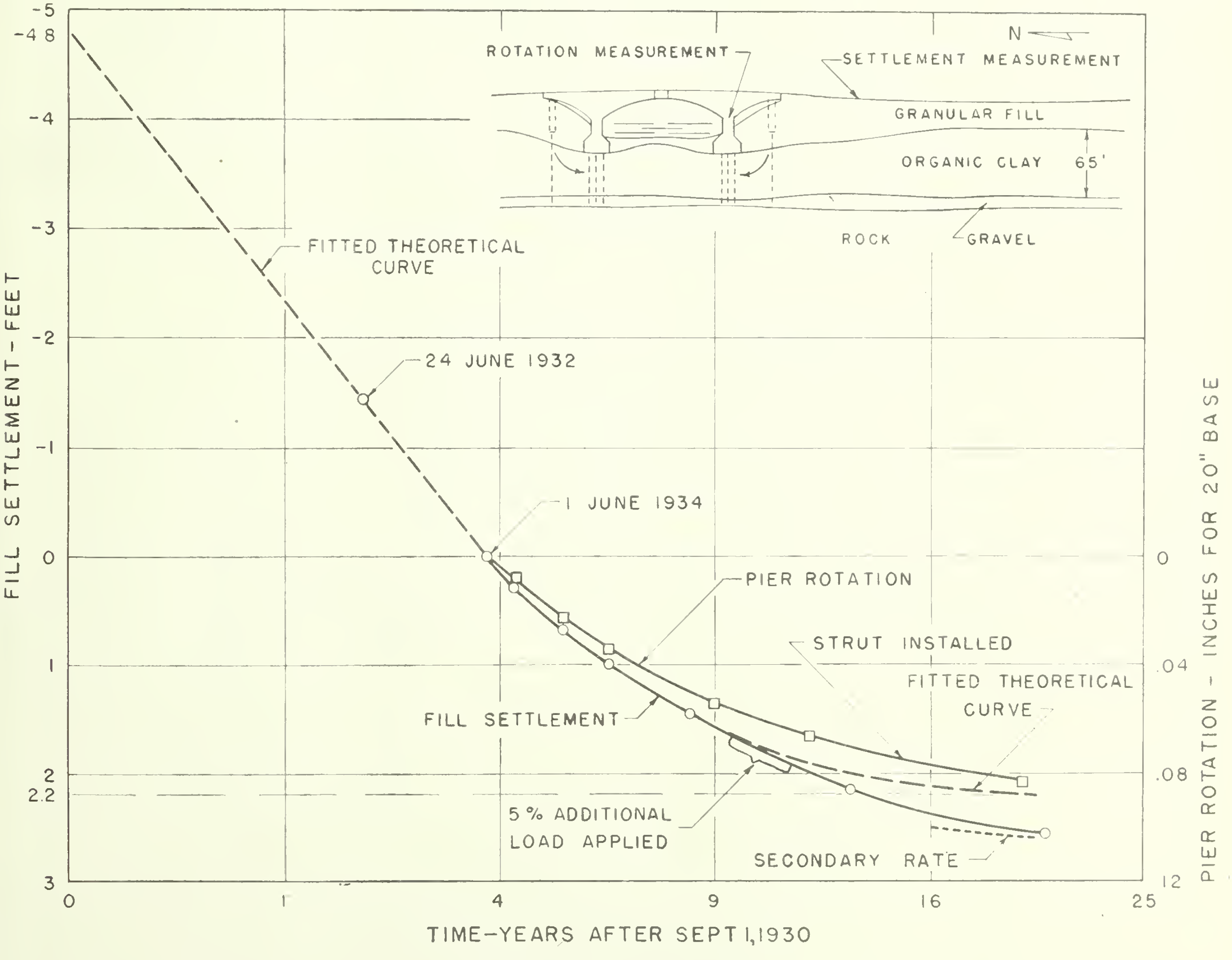

Figure 4.—Fill settlement and pier rotation at Boundary Channel Bridge.

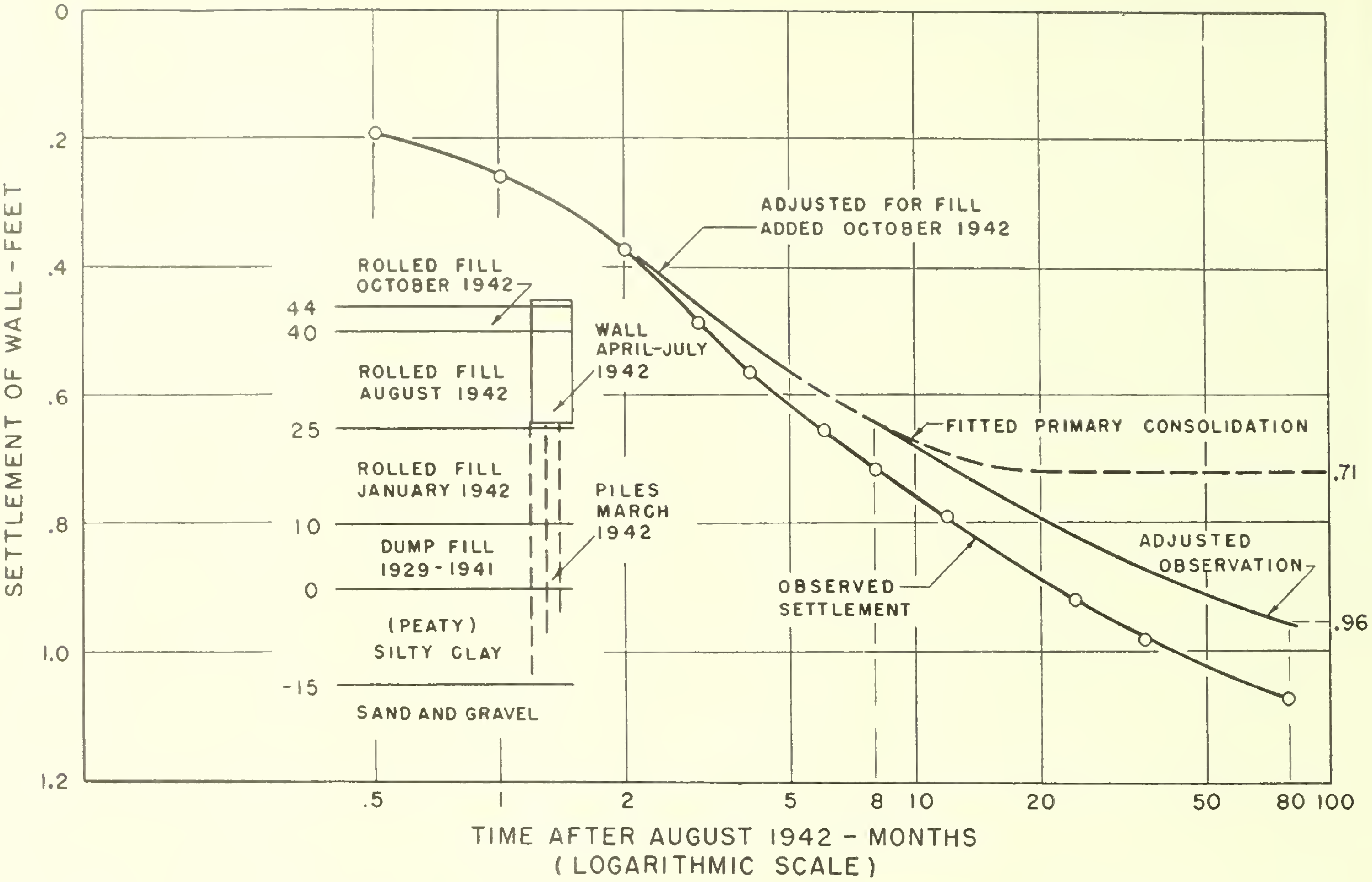

Figure 5.—Settlement of north-east wing wall, Bridge 8, Pentagon network.

Table 5.—*Consolidation properties of clay at Boundary Channel Bridge*

Sample	Coefficient of consolidation c_v	Compressibility m	$\frac{1}{c_v m}$
	Ft. sq./day	*Sq. ft./kip*	
3A	0.35	0.049	58
3B	.03	.059	566
4	.28	.046	78
5	.07	.033	430
6	.14	.046	155
11	.08	.050	250
12	1	.020	50
Total	1.95	.303	1,587
Average	.28	.043	229

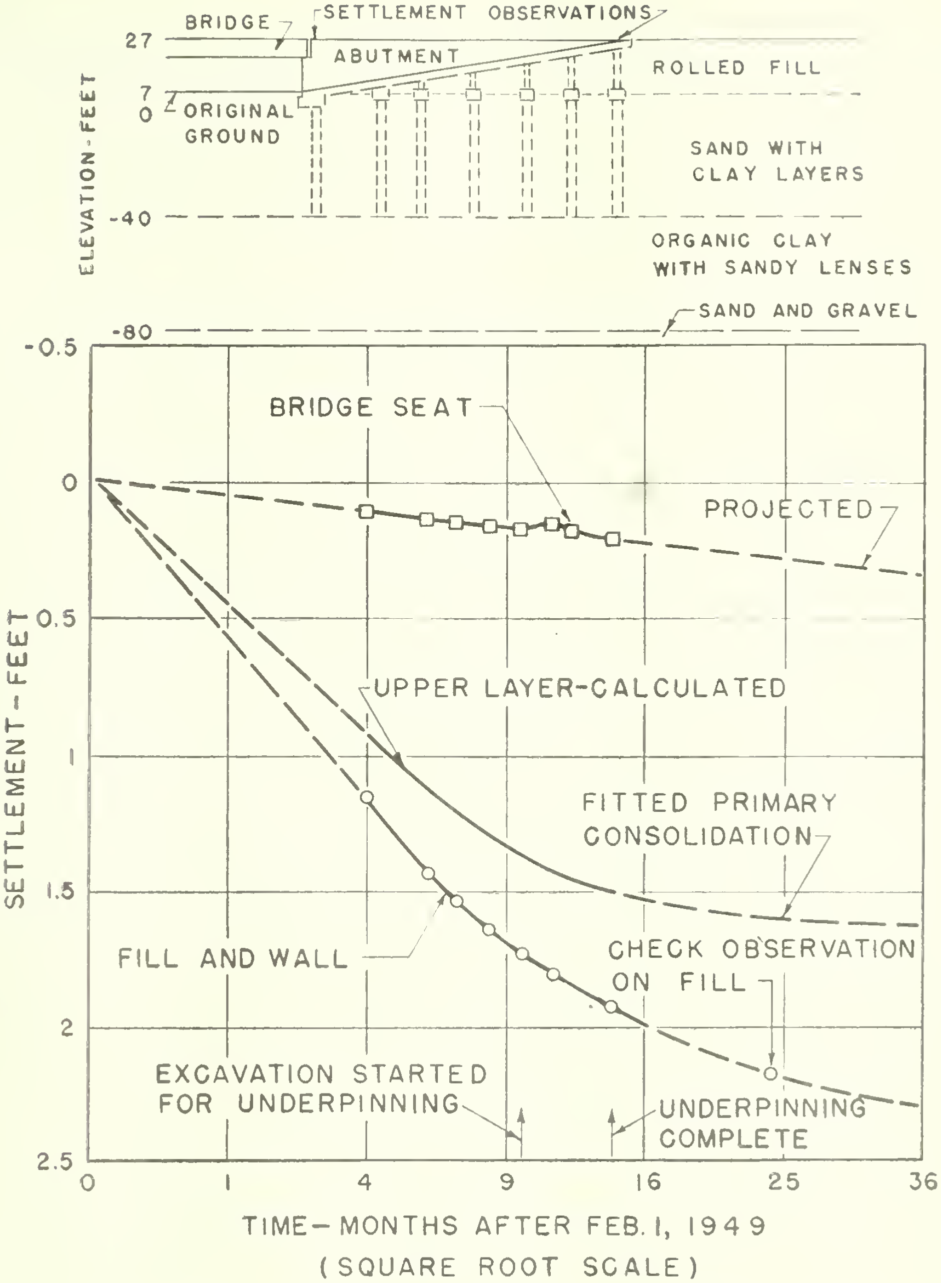

Figure 6.—Settlement at north abutment of new Fourteenth Street Bridge.

Second Inter-American Highway Film

Inter-American Highway Report — Part II, Central America and Panama, a motion picture produced by the Bureau of Public Roads, is now available for lending to interested organizations. The 16-millimeter sound and color film, with a running time of 62 minutes, shows the present condition of the southerly 1,600 miles of the Inter-American Highway extending from the Guatemala-Mexico boundary through the Central American Republics of Guatemala, El Salvador, Honduras, Nicaragua, and Costa Rica and thence through Panama to Panama City. *Inter-American Highway Report—Part I, Mexico*, covering the northern 1,700 miles of the highway, was announced in PUBLIC ROADS, vol. 26, No. 10, October 1951.

Inter-American Highway Report—Part II is a study in rich, colorful, and vivid contrasts. Portions of the route are in splendid condition. On others no work has been done and the cars of the motion-picture men had to be dragged across muddy quagmires and through deep river fords. There are modern, bustling capital cities to compare with primitive rural villages, ox carts to contrast with present-day motor-vehicle traffic, handicrafts which hark back to the earliest times side by side with twentieth-century industrial plants, and an ever-changing kaleidoscope of beautiful scenery, historic structures, ancient ruins and Indian temples, and beautiful cathedrals.

Animated charts show the bypass around the uncompleted section in northern Guatemala as well as the steamship journey on the bypass route from Costa Rica to Panama. Animated maps are flashed upon the screen at the conclusion of the pictures for each country. These maps locate the capitals and give the location and mileage of the passable and impassable sections.

Sections of modern highway, up-to-date bridges spanning wide rivers, garages, filling stations, haciendas, hotels, and all the other attributes of a main route, which will some day make touring through Central America a must for the motorist, appear. The picture summarizes the present condition of this great thoroughfare—a unit in the greater Pan American Highway which will some day join North and South America—and gives an accurate appraisal of the work that still remains to be done before it will be possible for the casual motorist to essay the journey over the entire route.

Inter-American Highway Report — Part II, Central America and Panama, may be borrowed by any responsible organization, without cost except for the nominal transportation charges, by writing to the Visual Education Branch, Bureau of Public Roads, Washington 25, D. C.

Traffic Trends on Rural Roads in 1950

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by
THOMAS B. DIMMICK,
Head, Current Data Analysis Unit

Total travel on rural roads in 1950 broke all records, exceeding the 1949 previous high by 9 percent and the 1941 prewar peak by 38 percent. On the 350,000 miles of main rural roads in the United States, travel in 1950 was over 174 billion vehicle-miles, of which 76 percent was by passenger cars, 1 percent by busses, and 23 percent by freight-carrying vehicles.

Trucks and combinations hauled 36 percent more ton-mileage of freight in 1950 than in 1949 and 106 percent more than in 1941, the increase resulting largely from greater use of heavier vehicles. Truck combination travel was 33 percent higher than in 1949 and 145 percent higher than in 1941. Comparable figures for single-unit trucks were 12 and 43 percent. The average carried load for all trucks and combinations in 1950 was 10 percent above the average in 1949 and 55 percent above that in 1941.

In 1950 almost 7 percent of all trucks and combinations exceeded a State legal weight limit, and 19 percent of the combinations were illegally overloaded in some particular. In comparison with 1949, the percentage of overweight vehicles for 1950 increased in all regions except in the South Atlantic States.

MOTOR-VEHICLE TRAVEL broke all previous records in 1950 for the fifth consecutive year. The 1950 traffic on all rural roads was almost 9 percent higher than in 1949, 18 percent higher than in 1948, 26 percent higher than in 1947, almost 38 percent higher than in 1946, and slightly more than 38 percent higher than the 1941 prewar peak. Geographically, the increases over 1949 ranged from 7 percent in the western States to 8 percent in the eastern States and 10 percent in the central States. The largest increase over 1949 in any of the United States census regions[1] was 15 percent in the East South Central region. The smallest increase was 4 percent in the Pacific region. Records from about 900 automatic traffic recorders, operated continuously throughout the year at permanent stations on main and local roads in all States, were used generally to establish these trends. More extensive traffic surveys, made by a number of States, yielded valuable information concerning the total volume of rural traffic within their boundaries. Consideration has been given to all such available data in this analysis. Where States have prepared and submitted vehicle-mile travel estimates of their own, these have been employed rather than estimates made by applying trend factors.

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[1] The States comprising each census region are indicated in table 1.

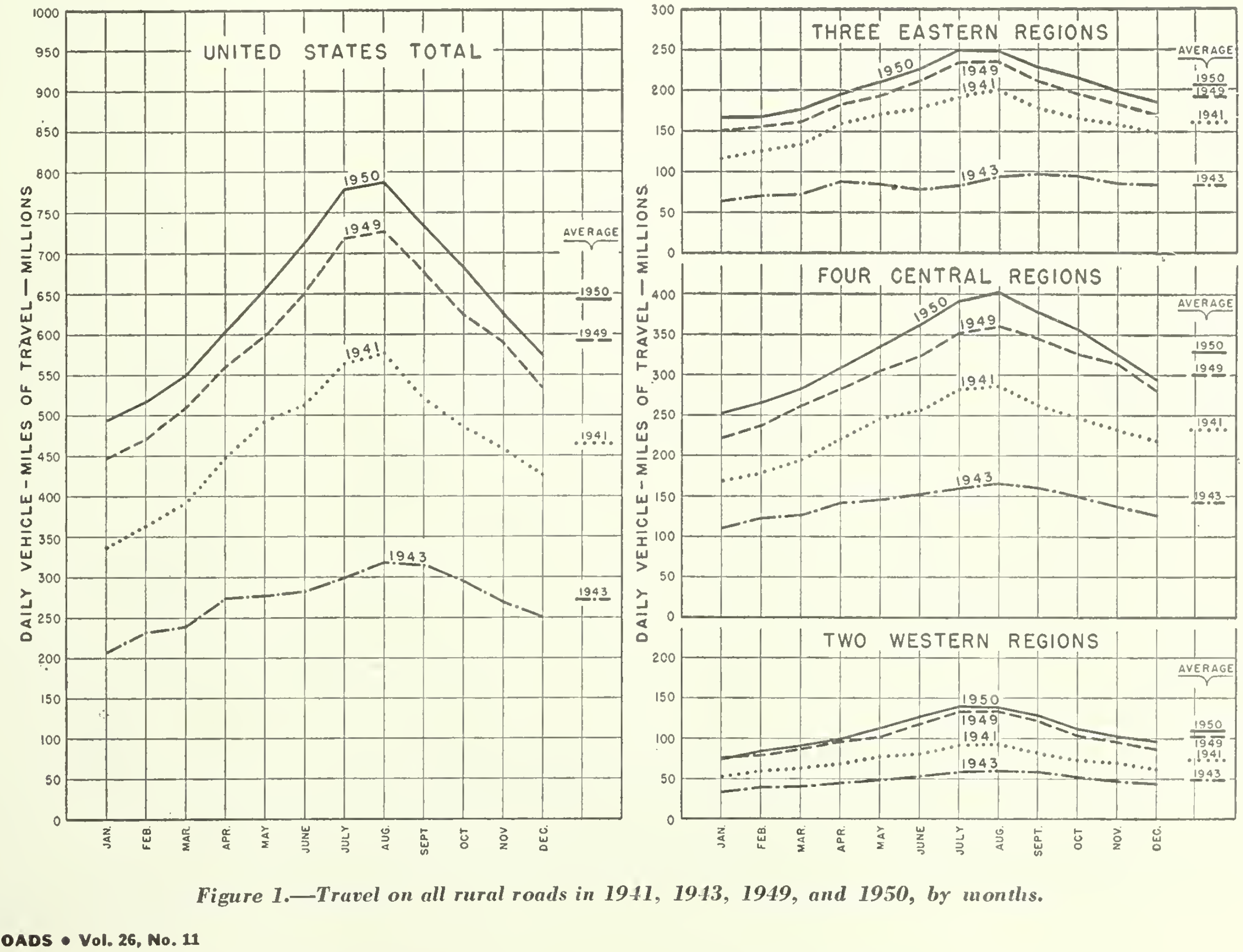

Figure 1.—Travel on all rural roads in 1941, 1943, 1949, and 1950, by months.

The variation in average daily travel on rural roads by months in the three main geographic divisions and in the United States as a whole is illustrated in figure 1 for the years 1950, 1949, 1943, and 1941, the latter being the prewar peak year. Travel in each month of 1950 in the eastern and central regions and in the United States as a whole was well above that of the corresponding month of the earlier years. The western region showed a slight decrease in January from 1949 to 1950.

Summer travel constituted a greater portion of the annual travel in 1950 than in any recent year. In the last two prewar years (1940 and 1941) the average daily traffic in July and August was 23 percent above the average daily traffic for the year. During the war this seasonal travel was reduced drastically, the average daily traffic in July and August being only 13 percent above the annual average in 1942 and 1943. Not until 1949 did vacation and other summer driving form as large a proportion of the year's travel as in the prewar years. In 1950 the average daily traffic on rural roads in July and August was slightly more than 24 percent above the annual average, a percentage even higher than in 1940 and 1941.

Table 1.—*Survey period, number of stations operated, number of vehicles counted, and number weighed in each State in the special weight surveys, summer of 1950*

Region and State	Survey period	Number of stations	Total vehicles counted	Trucks and truck combinations	
				Counted	Weighed
New England:					
Connecticut	July 24–Aug. 17	10	33,055	6,611	2,135
Maine	July 26–Aug. 11	10	27,710	5,453	2,311
Massachusetts	Aug. 7–Aug. 22	10	39,757	7,108	2,448
New Hampshire	Aug. 7–Aug. 11	5	14,753	2,061	610
Rhode Island	July 17–July 21	5	13,881	2,643	1,202
Vermont	July 31–Aug. 4	5	9,860	775	775
Subtotal		45	139,016	24,651	9,481
Middle Atlantic:					
New Jersey	Aug. 14–Aug. 29	10	83,027	16,354	1,590
New York	Aug. 1–Sept. 5	20	33,536	8,419	1,747
Pennsylvania	July 19–Sept. 21	14	56,742	13,042	2,680
Subtotal		44	173,305	37,815	6,017
South Atlantic:					
Delaware	Aug. 10–Aug. 23	9	40,861	8,547	1,443
Florida	No survey				
Georgia	Aug. 8–Oct. 24	18	25,107	6,701	3,485
Maryland	Aug. 14–Sept. 13	10	42,975	9,570	1,236
North Carolina	Aug. 8–Aug. 29	12	29,462	7,194	4,276
South Carolina	Sept. 11–Sept. 22	10	19,181	5,368	2,031
Virginia	Aug. 2–Aug. 15	10	25,030	6,003	3,926
West Virginia	Aug. 8–Aug. 24	9	14,381	3,762	1,418
Subtotal		78	196,997	47,145	17,815
Eastern regions, subtotal		167	509,318	109,611	33,313
East North Central:					
Illinois	No survey				
Indiana	Aug. 2–Aug. 29	20	51,375	12,849	5,295
Michigan	June 13–June 27	9	22,620	4,665	1,682
Ohio	July 18–Aug. 3	10	28,841	5,599	1,357
Wisconsin	Aug. 1–Sept. 29	10	24,502	4,879	1,080
Subtotal		49	127,338	27,992	9,414
East South Central:					
Alabama	July 18–Aug. 25	10	13,318	3,290	2,495
Kentucky	Aug. 2–Sept. 14	6	9,532	2,741	1,222
Mississippi	July 10–July 28	15	25,197	6,635	3,617
Tennessee	Aug. 1–Sept. 7	10	13,749	3,845	1,472
Subtotal		41	61,796	16,511	8,806
West North Central:					
Iowa	July 24–Aug. 23	10	13,937	2,679	2,674
Kansas	July 17–Aug. 12	10	11,089	2,263	993
Minnesota	July 10–Aug. 25	19	23,238	3,860	1,906
Missouri	July 31–Aug. 28	16	139,548	27,732	9,820
Nebraska	July 20–Aug. 29	20	24,011	5,250	5,157
North Dakota	July 20–Aug. 30	14	22,689	5,233	2,213
South Dakota	June 23–Sept. 20	11	9,985	1,350	1,149
Subtotal		100	244,497	48,367	23,912
West South Central:					
Arkansas	Sept. 11–Sept. 29	10	18,638	6,333	1,523
Louisiana	July 31–Aug. 4	10	11,769	3,460	925
Oklahoma	July 17–Aug. 14	10	15,512	3,443	3,225
Texas	June 1–Aug. 31	20	98,441	21,032	5,372
Subtotal		50	144,360	34,268	11,045
Central regions, subtotal		240	577,991	127,138	53,177
Mountain:					
Arizona	July 10–July 21	10	9,923	2,043	857
Colorado	Aug. 3–Aug. 22	13	26,180	4,266	915
Idaho	No survey				
Montana	July 31–Sept. 1	9	9,477	1,921	1,108
Nevada	Aug. 1–Aug. 18	10	7,613	1,084	988
New Mexico	July 31–Aug. 14	10	14,371	3,251	1,467
Utah	July 7–Aug. 4	10	18,954	3,595	1,306
Wyoming	July 31–Aug. 18	10	12,625	2,386	891
Subtotal		72	99,143	18,546	7,532
Pacific:					
California	May 31–July 7	20	[1] 80,740	14,855	5,117
Oregon	Aug. 8–Sept. 1	8	16,456	3,251	2,179
Washington	June 5–Oct. 10	20	97,088	17,373	12,984
Subtotal		48	194,284	35,479	20,280
Western regions, subtotal		120	293,427	54,025	27,812
United States total		527	1,380,736	290,774	114,302

[1] Passenger cars not counted; figure given is an estimate based on data from other reports.

Source of Information

The large number of automatic traffic recorders operated on the rural roads of each State give a good indication of the trend of total traffic on those highways but provide no indication of the classification of vehicles by type, weight, or other characteristics. During certain prewar years, generally 1936 or 1937, nearly every State conducted a comprehensive survey of traffic in which all vehicles counted were classified by type. At the same time a large number of trucks and truck combinations were stopped and information recorded concerning their weight, dimensions, and other important features.

In order to determine the wartime trend in weights, dimensions, and other characteristics of commercial vehicles, a brief check survey was made in the summer of 1942 at certain typical stations in most States. From strictly comparable information gathered in the two surveys, trends were calculated which were used to determine the changes in traffic and vehicle characteristics that had taken place since the comprehensive survey was made. Since 1942, check surveys have been made annually. Most States have participated in these each year and all have participated at some time.[2] Forty-five States conducted such surveys in 1950.

Classification counts made in numerous States, in addition to those made at the weight stations, added valuable information concerning vehicle-type proportions. In a few States greatly expanded loadometer surveys have furnished more reliable data concerning vehicle types and weights than can be obtained from the trend data alone, and these have been used in the analysis when available.

1950 Summer Loadometer Survey

The stations used in these check surveys were selected initially to give a representative cross section of traffic on main rural roads. They were operated for one or more 8-hour periods on a weekday, generally from either 6 a. m. to 2 p. m., or from 2 p. m. to 10 p. m. All traffic passing through the stations during the period was counted and classified into the following categories: Local passenger cars; foreign (out-of-State) passenger cars; panel and pick-up trucks;[3] other two-axle, four-tire trucks; two-axle, six-tire trucks; three-axle trucks; truck-tractor and semitrailer combinations; truck and trailer combinations or truck-tractor semitrailer and trailer combinations; and busses. The combination-type vehicles were further subdivided according to the number of axles of each.[4]

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[2] See *Traffic trends on rural roads*, by T. B. Dimmick, PUBLIC ROADS, vol. 26, No. 5, Dec. 1950; vol. 25, No. 12, Feb. 1950; vol. 25, No. 7, Mar. 1949; vol. 25, No. 3, Mar. 1948; vol. 24, No. 10, Oct.-Nov.-Dec. 1946; and *Amount and characteristics of trucking on rural roads*, by J. T. Lynch and T. B. Dimmick, PUBLIC ROADS, vol. 23, No. 9, July-Aug.-Sept. 1943.

Most of the weight stations were operated during July, August, and September. The survey period, number of stations operated, number of vehicles counted, and number weighed are shown for each State in table 1. More than 1.38 million vehicles were counted at all stations during the period of the survey. Slightly more than one-fifth of these were freight-carrying vehicles, of which almost 40 percent were weighed.

Wherever traffic volume permitted, every truck and truck combination was stopped and weighed. Where this procedure was impracticable all of the less common types were weighed and the common vehicle types were weighed in sufficient numbers to establish their characteristics from the sample. The type of vehicle, whether loaded or empty, the number of axles, and the weight of each axle were recorded. The axle-spacing and total wheelbase length of the heavier vehicles[5] were measured, and the commodity carried and the type of operation—private or for-hire—were recorded. Passenger cars and busses were counted but not stopped for weighing.

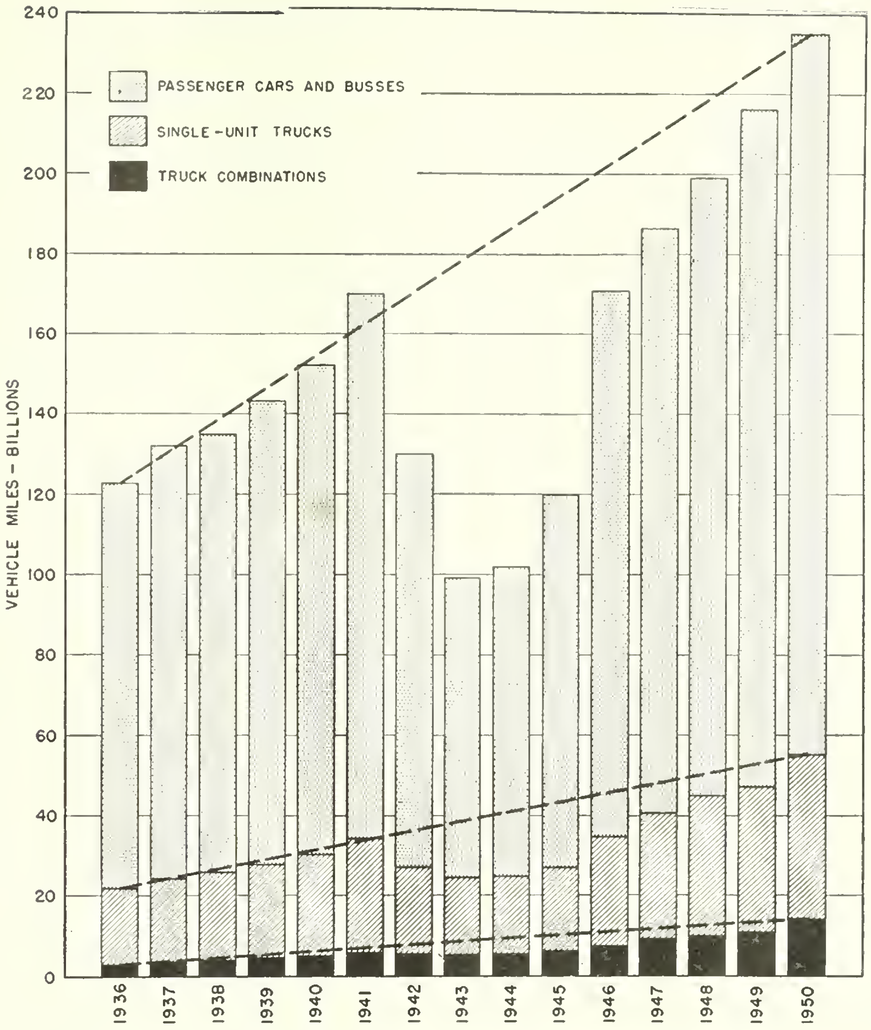

Figure 2.—Travel on all rural roads, 1936-50, by classes of vehicles.

Prewar Traffic Trend Increased

Figure 2 shows in chart form the vehicle-mileage of travel on all rural roads, by types, for each year from 1936 to 1950, inclusive.[6] It is apparent that the effect of the drastic restrictions of travel during the war period, 1942–45, caused but a temporary dip in traffic growth and that the 1950 vehicle-mileage was as high as would have been estimated by any rational projection of the prewar trend. A straight line from the top of the bar for 1936 to the top of the bar for 1950 passes through the top of the bar for 1937, cuts below the top of the bar for 1941, and falls well above the tops of the bars for all other years.

Travel by trucks and truck combinations increased in a manner very similar to that observed for all vehicles. For truck combinations alone, the 1936–50 line lies above the tops of all bars from 1937 to 1949, inclusive, thus showing an accelerating upward trend in the travel by these heavier vehicles. This is emphasized by other trend data, given in other portions of this report.

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[3] Single-unit trucks with a carrying capacity of less than 1½ tons.

[4] In this article, the term "truck" is used to indicate a single-unit vehicle; "truck combination" to indicate truck-tractor semitrailer (with or without full trailer) and truck with full trailer; and "trucks and truck combinations" or "trucks and combinations" to indicate all of these vehicles together.

[5] Trucks and truck combinations weighing 13 tons or more, or having an axle weighing 18,000 pounds or more.

[6] In a similar figure in *Traffic trends on rural roads in 1949*, PUBLIC ROADS, vol. 26, No. 5, Dec. 1950, the bar for 1938 was shorter than it should have been. The current figure is correctly plotted.

Table 2.—*Ratio of 1950 traffic on main rural roads to corresponding traffic in 1949* [1]

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
Passenger cars:													
Local	1.07	1.03	1.11	1.07	1.02	1.11	1.07	1.13	1.07	1.18	1.03	1.04	1.06
Foreign	1.04	1.06	1.11	1.08	1.19	1.16	1.10	1.13	1.16	1.08	1.02	1.06	1.11
All passenger cars	1.06	1.04	1.11	1.07	1.07	1.12	1.07	1.13	1.09	1.13	1.03	1.04	1.07
Trucks and combinations:													
Single-unit trucks	1.16	1.09	1.10	1.11	1.08	1.16	1.09	1.12	1.12	1.10	1.13	1.12	1.12
Truck combinations	1.32	1.23	1.34	1.31	1.39	1.34	1.14	1.22	1.32	1.28	1.36	1.35	1.33
All trucks and combinations	1.19	1.14	1.16	1.16	1.19	1.20	1.10	1.14	1.18	1.13	1.19	1.19	1.18
Busses	1.03	.78	.97	.91	1.17	1.02	1.04	.99	1.06	1.00	1.01	1.01	.99
All vehicles	1.08	1.06	1.12	1.09	1.09	1.14	1.08	1.13	1.11	1.13	1.06	1.07	1.09

[1] The ratios for "all vehicles" are based on year-around automatic recorder data, while those for the individual vehicle types are based principally on summer counts.

Table 3.—*Percentage distribution of travel, by vehicle type and by type of operation, on main rural roads in the summer of 1950*

Vehicle type	Eastern regions				Central regions					Western regions			United States average	U. S. percentage distribution of trucks and truck combinations by type of operation		
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average		Total	Private	For-hire
Passenger cars:																
Local	55.41	61.97	56.66	58.66	54.73	43.88	59.04	58.74	55.17	43.58	70.97	60.66	57.26	......	......	......
Foreign	24.39	16.17	18.83	18.51	22.64	22.76	15.64	14.99	19.19	32.75	10.33	18.77	18.89	......	......	......
All passenger cars	79.80	78.14	75.49	77.17	77.37	66.64	74.68	73.73	74.36	76.33	81.30	79.43	76.15	......	......	......
Single-unit trucks:																
Panel and pick-up	4.68	4.91	7.52	6.05	5.19	11.93	8.10	10.56	8.12	9.73	4.54	6.49	7.16	31.35	42.38	1.91
Other 2-axle, 4-tire	1.36	.88	.51	.78	.49	.51	.77	.33	.51	.66	.79	.74	.64	2.79	3.70	.37
Other 2-axle, 6-tire	8.04	7.82	7.46	7.69	6.48	12.22	9.21	7.19	8.08	6.22	4.53	5.17	7.45	32.63	36.72	21.68
3-axle	.32	.32	.40	.36	.40	.38	.28	.17	.31	.45	.72	.62	.38	1.68	1.54	2.02
All single-unit trucks	14.40	13.93	15.89	14.88	12.56	25.04	18.36	18.25	17.02	17.06	10.58	13.02	15.63	68.45	84.34	25.98
Truck-tractor and semitrailer combinations:																
3-axle	4.27	5.78	5.01	5.22	5.17	5.53	3.13	4.40	4.57	1.87	.98	1.31	4.23	18.50	10.16	40.79
4-axle	.27	1.33	2.34	1.64	3.24	1.09	2.40	2.41	2.54	1.54	1.36	1.43	2.05	8.99	4.01	22.31
5-axle or more	......	.01	.01	.01	.16	.02	.34	.06	.16	1.21	2.64	2.10	.44	1.94	.61	5.49
All truck-tractor and semitrailer combinations	4.54	7.12	7.36	6.87	8.57	6.64	5.87	6.87	7.27	4.62	4.98	4.84	6.72	29.43	14.78	68.59
Truck and trailer combinations:																
4-axle or less	.03	.02	.02	.02	.16	......	.25	.17	.16	.27	.48	.40	.16	.68	.48	1.21
5-axle	......	.01	(1)	.01	.46	......	.01	.01	.18	.33	.68	.55	.19	.82	.19	2.49
6-axle or more	......	......	(1)	(1)	.09	......	......	......	.04	.26	.99	.72	.14	.62	.21	1.73
All truck and trailer combinations	.03	.03	.02	.03	.71	......	.26	.18	.38	.86	2.15	1.67	.49	2.12	.88	5.43
All combinations	4.57	7.15	7.38	6.90	9.28	6.64	6.13	7.05	7.65	5.48	7.13	6.51	7.21	31.55	15.66	74.02
All trucks and truck combinations	18.97	21.08	23.27	21.78	21.84	31.68	24.49	25.30	24.67	22.54	17.71	19.53	22.84	100.00	100.00	100.00
Busses	1.23	.78	1.24	1.05	.79	1.68	.83	.97	.97	1.13	.99	1.04	1.01	......	......	......
All vehicles	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	......	......	......

[1] Less than 0.005 percent.

Travel Increases

The ratio of traffic volumes on main rural roads in 1950 to the corresponding volumes in 1949 is shown in table 2. Highways classified under the term "main" include about 350,000 miles and, in general, are those of the entire State systems. In such States as North Carolina, Pennsylvania, and Virginia, where all or a large part of the rural-road mileage is under State control, only the mileage in the State primary system is included. The consistent increase in travel on these main highways by most types of vehicles and in all sections of the country is evident in the table.

Travel by both local and foreign (out-of-State) passenger cars, single-unit trucks, and truck combinations increased in all regions. Travel by busses, however, decreased or remained about the same in four regions, declining slightly for the United States as a whole. In general, travel by out-of-State passenger cars increased more than that by local passenger cars, reflecting a higher rate of increase for tourist travel, which is consistent with the increased percentage for the summer peak, already noted.

The increase in travel by all types of freight-carrying vehicles amounted to 18 percent, compared to 7 percent for passenger cars. Truck registrations increased only 7 percent, and greater use of the registered vehicles is therefore indicated. Perhaps the most significant fact shown by table 2 is that travel by truck combinations increased much faster than travel by single-unit trucks, the increase by these heavier vehicles amounting to 33 percent.

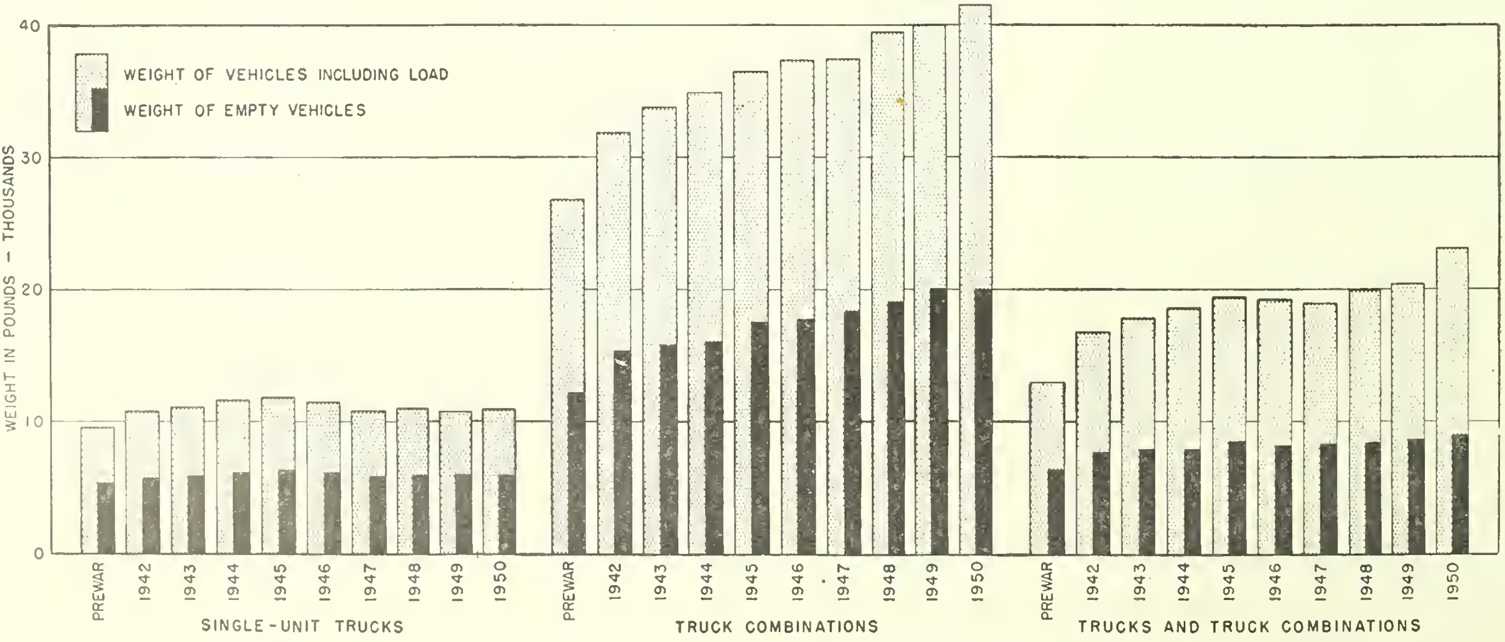

Figure 3.—*Average weights of loaded and of empty trucks and truck combinations in the summers of 1942-50 and a prewar year.*

Table 4.—*Average weights (in pounds) of loaded and empty trucks and truck combinations, by vehicle types, in the summer of 1950*

Vehicle type	Eastern regions				Central regions					Western regions			United States average	U. S. average by type of operation	
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average		Private	For-hire
AVERAGE WEIGHTS OF LOADED VEHICLES															
Single-unit trucks:															
Panel and pick-up	4,950	5,356	4,805	5,038	4,822	5,373	5,105	7,129	5,735	5,154	4,351	4,695	5,370	5,367	5,467
Other 2-axle, 4-tire	6,419	8,072	6,436	7,003	6,737	7,295	7,883	6,720	7,177	7,144	5,871	6,148	6,868	6,701	10,419
Other 2-axle, 6-tire	14,577	15,581	13,007	14,288	13,167	14,575	14,025	13,405	13,712	14,184	12,815	13,351	13,853	13,319	16,375
3-axle	29,566	31,246	28,193	29,426	27,291	26,736	26,770	27,252	27,073	32,279	26,552	27,748	27,939	26,974	29,790
Average	11,607	12,540	10,536	11,490	10,505	11,979	10,841	10,131	10,739	10,534	10,219	10,342	10,902	10,118	16,574
Truck combinations:															
Truck-tractor and semitrailer	38,666	41,731	38,175	39,687	38,888	35,068	40,495	38,316	38,612	46,859	51,078	49,721	40,557	39,225	41,231
Truck and trailer	(1)	57,897	(1)	43,308	64,466	....	25,365	34,497	54,572	63,154	56,069	57,259	56,111	42,692	62,694
Average	38,487	41,802	38,169	39,699	40,374	35,068	39,935	38,231	39,257	49,013	52,393	51,423	41,511	39,431	42,569
Average, all trucks and combinations	19,539	24,615	22,233	22,851	25,323	19,217	19,895	20,095	22,009	24,013	29,358	27,526	23,188	16,155	36,938
AVERAGE WEIGHTS OF EMPTY VEHICLES															
Single-unit trucks:															
Panel and pick-up	4,166	4,329	3,783	3,984	3,802	4,050	4,080	4,863	4,236	4,063	3,746	3,956	4,122	4,121	4,185
Other 2-axle, 4-tire	5,095	4,951	5,081	5,012	4,952	5,637	5,743	5,126	5,364	5,108	4,603	4,879	5,128	5,078	7,232
Other 2-axle, 6-tire	8,506	8,944	7,294	8,170	7,740	7,749	7,970	7,952	7,848	8,111	7,911	8,023	7,979	7,856	8,455
3-axle	15,371	16,903	13,526	15,063	13,731	9,483	15,469	16,957	13,802	15,377	13,896	14,521	14,406	14,152	14,871
Average	6,655	6,976	5,324	6,100	5,880	5,586	5,928	6,046	5,861	5,477	5,757	5,583	5,904	5,640	8,593
Truck combinations:															
Truck-tractor and semitrailer	20,391	20,348	18,843	19,656	18,587	17,339	20,586	18,971	18,877	23,833	23,530	23,683	19,555	19,265	19,731
Truck and trailer	(1)	24,144	(1)	20,050	25,611	....	13,232	20,997	23,390	29,074	27,513	27,906	25,601	21,199	27,176
Average	20,392	20,369	18,829	19,658	19,447	17,339	20,215	19,035	19,190	24,960	25,338	25,181	20,043	19,364	20,483
Average, all trucks and combinations	9,067	10,422	8,142	9,135	10,147	7,190	8,505	8,650	8,719	8,271	11,183	9,499	8,953	7,135	16,336

[1] Data omitted because of insufficient sample.

Use of Truck Combinations

The percentage of travel by vehicle types on main rural roads in 1950 is given in table 3. In this table all single-unit trucks are divided into classification types based on the axle and tire arrangements, while the truck combinations are classified according to the total number of axles of the combination. The classification of vehicles into these types has been used only in the last four annual surveys. It has several advantages over the old "light, medium, and heavy" grouping, particularly in that it provides more homogeneous groupings and more positive identification of the types. It is regrettable that no direct comparison can be made by vehicle types between the old and the new classifications, or between data collected in 1946 and earlier years with such data collected in 1947 and thereafter, but the convenience and advantages of the new system outweigh the disadvantages caused by the change.

The data in table 3 indicate that in 1950 truck and truck combination travel was more than 20 percent of the total travel in all but the New England and Pacific regions. It was between 20 and 26 percent in all of the remaining regions except the East South Central region, where it was well over 30 percent.

A comparison with the same table in the 1949 report shows that the proportion of trucks was higher in 1950 than in 1949 in every region except the Mountain region, where it remained about the same.

The table indicates that the usage of certain types of freight-carrying vehicles varies in different sections. For instance, the truck and trailer combinations with six or more axles and the truck-tractor and semi-

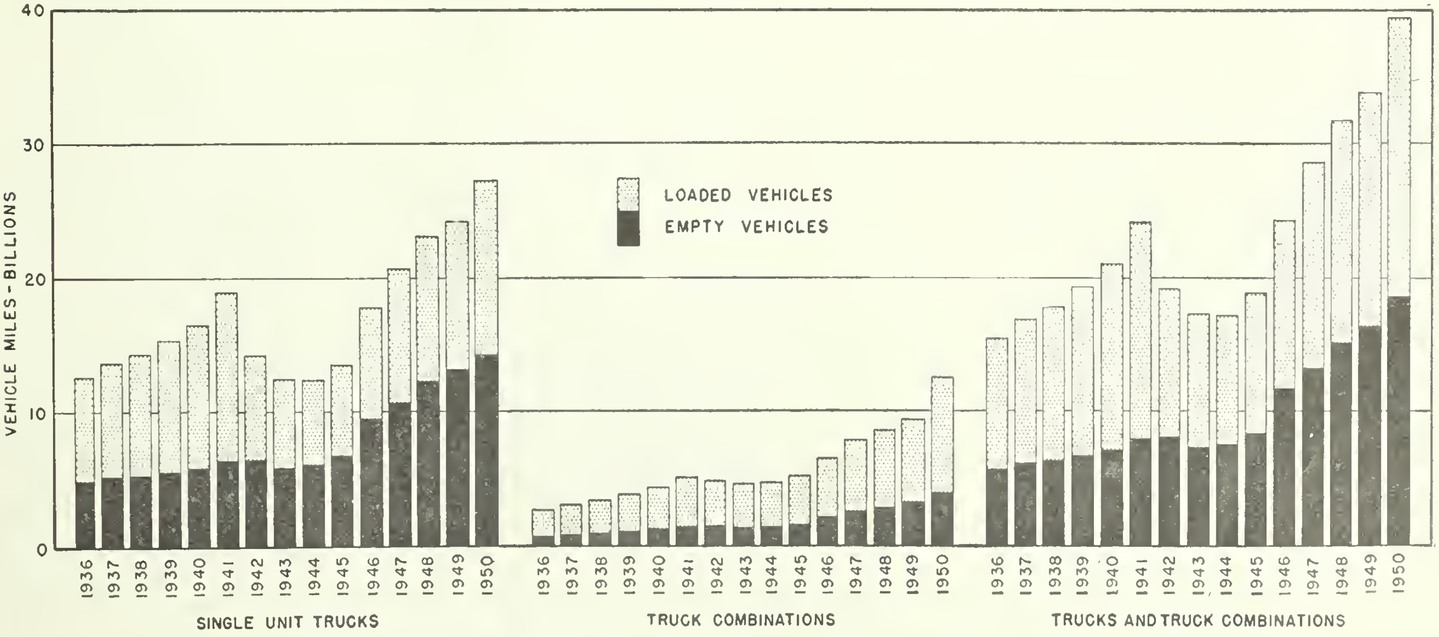

Figure 4.—Travel on main rural roads, 1936-50, by loaded and by empty trucks and truck combinations.

Table 5.—*Comparison of estimated vehicle-miles of travel on main rural roads in 1936, 1941, 1946, 1949, and 1950*

Year	All vehicles, vehicle-miles	Passenger cars and busses [1]		All trucks and truck combinations		Single-unit trucks		Truck combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles
	Millions		*Millions*		*Millions*		*Millions*		*Millions*
1936	88,412	82.6	73,005	17.4	15,407	82.1	12,650	17.9	2,757
1941	122,505	80.3	98,320	19.7	24,185	78.8	19,057	21.2	5,128
1941:1936 ratio	*1.39*	*.97*	*1.35*	*1.13*	*1.57*	*.96*	*1.51*	*1.18*	*1.86*
1946	124,149	80.4	99,803	19.6	24,346	73.3	17,838	26.7	6,508
1946:1941 ratio	*1.01*	*1.00*	*1.02*	*.99*	*1.01*	*.93*	*.94*	*1.26*	*1.27*
1946:1936 ratio	*1.40*	*.97*	*1.37*	*1.13*	*1.58*	*.89*	*1.41*	*1.49*	*2.36*
1949	159,379	78.8	125,602	21.2	33,777	71.9	24,295	28.1	9,482
1950	174,349	77.2	134,528	22.8	39,821	68.4	27,256	31.6	12,565
1950:1949 ratio	*1.09*	*.98*	*1.07*	*1.08*	*1.18*	*.95*	*1.12*	*1.12*	*1.33*
1950:1941 ratio	*1.42*	*.96*	*1.37*	*1.16*	*1.65*	*.87*	*1.43*	*1.49*	*2.45*
1950:1936 ratio	*1.97*	*.93*	*1.84*	*1.31*	*2.58*	*.83*	*2.15*	*1.77*	*4.56*
TRUCKS AND TRUCK COMBINATIONS IN PRIVATE OPERATION [2]									
1936	....	....	....	78.8	12,140	86.7	10,963	42.7	1,177
1949	....	....	....	77.2	26,077	91.6	22,262	40.2	3,815
1950	....	....	....	72.8	28,974	89.7	24,438	36.1	4,536
1950:1949 ratio	....	....	....	*.94*	*1.11*	*.98*	*1.10*	*.90*	*1.19*
1950:1936 ratio	....	....	....	*.92*	*2.36*	*1.03*	*2.23*	*.85*	*3.85*
TRUCKS AND TRUCK COMBINATIONS IN FOR-HIRE OPERATION [2]									
1936	....	....	....	21.2	3,267	13.3	1,687	57.3	1,580
1949	....	....	....	22.8	7,700	8.4	2,033	59.8	5,667
1950	....	....	....	27.2	10,847	10.3	2,818	63.9	8,029
1950:1949 ratio	....	....	....	*1.19*	*1.41*	*1.23*	*1.39*	*1.07*	*1.42*
1950:1936 ratio	....	....	....	*1.28*	*3.32*	*.77*	*1.67*	*1.12*	*5.08*

[1] Percentages of total 1950 travel by passenger cars and by busses are reported separately in table 3.

[2] The percentages below are percentages of the total number of type of vehicle indicated in the uppermost column head. For example, 86.7 percent of all single-unit trucks in 1936 were in private use.

trailer combinations with five or more axles are used far more frequently in the Pacific region than in any other area. Combinations involving trailers are used much less in the East South Central region and in the three eastern regions than in other sections. The use of combination-type vehicles has increased steadily in all regions in the last 5 years, the Nation-wide percentages of total travel being 7.21 in 1950, 5.95 in 1949, 5.84 in 1948, 5.73 in 1947, and 5.26 in 1946.

Private and For-Hire Traffic

In the survey conducted in 1950 information was gathered in most of the participating States concerning the use classification under which each vehicle was being operated. The data were reported separately for private and for-hire vehicles of each type, making possible the calculation of vehicle-mileages, ton-mileages, and other items concerning traffic on the main rural roads by the various types of trucks and truck combinations operated privately and operated for-hire.

In the last two columns of table 3 are shown the percentage distributions of private and for-hire trucks and combinations, by vehicle type. In general the lighter types of vehicles predominate in the private classification and, conversely, the heavier vehicles constitute a much higher proportion of the for-hire vehicles. This difference is especially marked in the percentages for the light panel and pick-up trucks and for the heavy combination-type vehicles. Over 42 percent of the privately operated trucks were of the panel and pick-up type, while less than 2 percent of the for-hire vehicles were of this type. On the other hand, less than 16 percent of the privately operated vehicles were truck combinations while 74 percent of the for-hire vehicles were combinations.

Average Weights Increase

The average weights of loaded and empty trucks and truck combinations, separately and combined, are shown graphically in figure 3 for each year from 1942 to 1950, inclusive, and for a prewar year, generally 1936 or 1937. The weights of single-unit trucks, both loaded and empty, increased each year from the 1936–37 period through 1945, then decreased somewhat or leveled off to an average amount slightly less than 11,000 pounds for loaded vehicles and slightly less than 6,000 pounds for empty vehicles. At the same time weights of truck combinations, both loaded and empty, have increased each year during the period shown. The increase in average weight of loaded combinations from the 1936–37 period to 1950 was over 55 percent, compared to 11 percent for single-unit trucks. The increase for all loaded trucks and truck combinations combined was 80 percent.

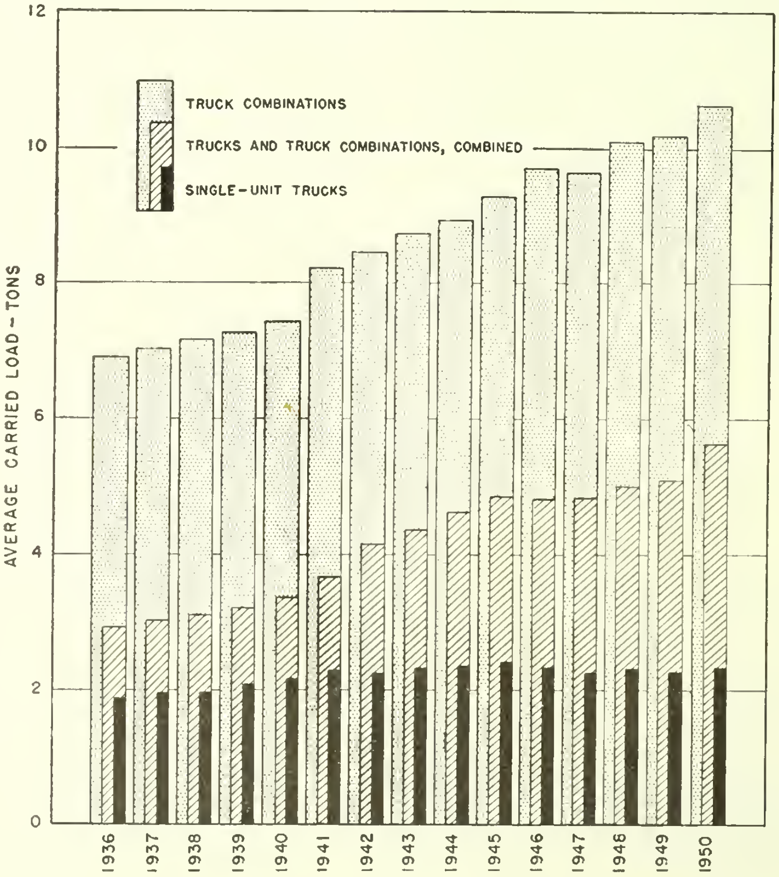

Figure 3.—Average load carried by trucks and truck combinations on main rural roads, 1936-1950.

It will be noted that the average weight of the loaded single-unit trucks was somewhat less than twice the average weight of the empty vehicles of this type, while the average weight of the loaded combinations was just about twice the average weight of the empty combinations. In the case of the vehicles of both types combined, the loaded vehicles included a higher proportion of combinations than the empty vehicles, since combinations are more often loaded, and the average weight of the loaded trucks and combinations was therefore considerably more than twice the average weight of the empty vehicles of both types.

The average weights of the various types of loaded and empty trucks and truck combinations in the summer of 1950 are shown in table 4 for the different regions. This table brings out clearly the important differences that exist in the weight characteristics of the vehicles in the different groups. It will be noted, for example, that for the United States as a whole, the loaded three-axle, single-unit trucks weighed about twice as much as the two-axle, six-tire trucks. The latter, in turn, weighed about twice as much as the two-axle, four-tire trucks. Similar differences existed throughout the various classifications. On the other hand, the regional differences in average weight for each of the vehicle types that are common throughout the country are surprisingly small. The rather low weights of truck and trailer combinations in the West North Central and West South Central regions indicate a predominance of small, home-made trailers of low capacity.

The average weights of loaded and empty trucks and truck combinations operated privately and for-hire in the summer of 1950 are shown in the last two columns of table 4. The for-hire vehicles, when compared by types, are generally heavier than those operated privately, and the average weight of all types of for-hire vehicles, either loaded or empty, is more than twice the average of the privately operated vehicles. It was shown in table 3 that most of the private vehicles consisted of small single-unit trucks while most of the for-hire vehicles consisted of the heavy truck combinations. This decided difference in the distributions of sizes of vehicles in the two operation classes accounts for the great difference between their average weights.

Truck Travel Increases

Figure 4 shows the estimated vehicle-mileage of travel by loaded and empty single-unit trucks and truck combinations, separately and combined, on main rural roads, for each year from 1936 to 1950, inclusive. This chart demonstrates graphically the steady growth of truck traffic during the prewar years 1936–41, the temporary effect of wartime restrictions in the period 1942–45, and the remarkable increases in truck transportation that have occurred since the end of hostilities in 1945.

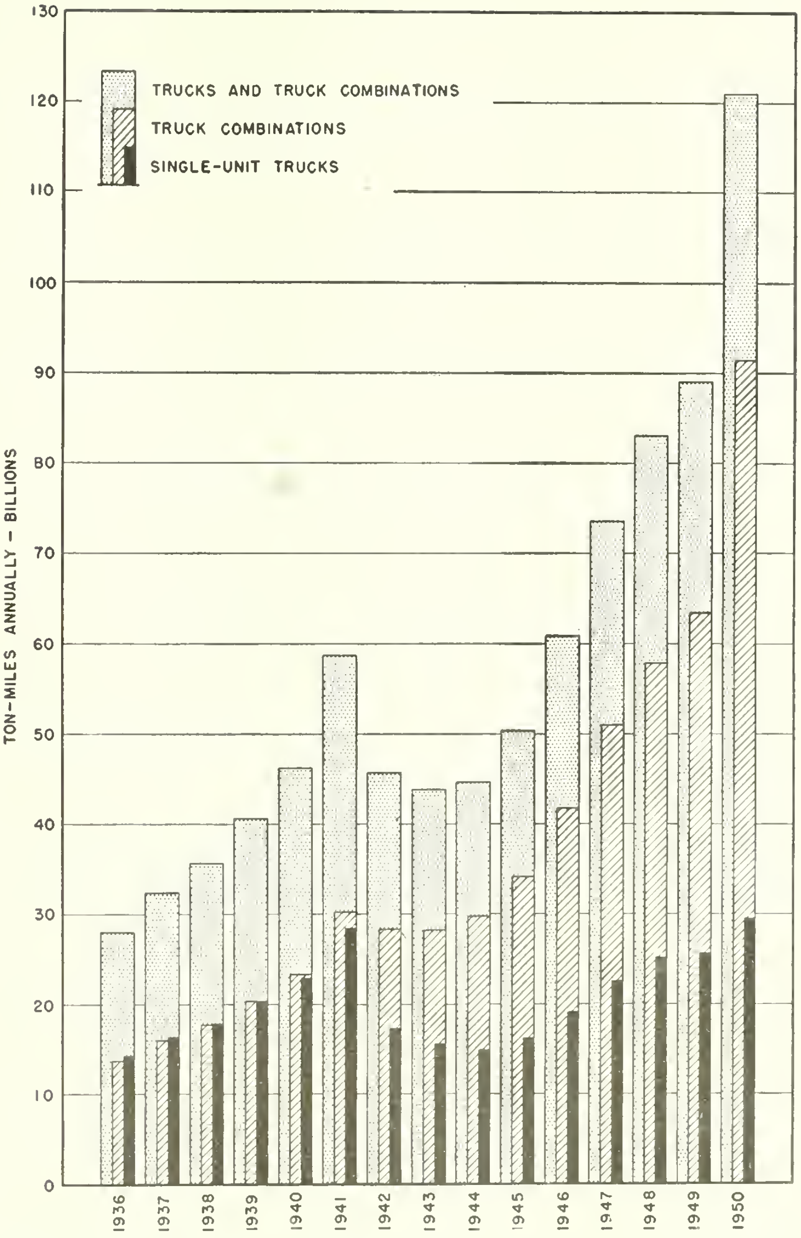

Figure 6.—Ton-miles carried by trucks and truck combinations on main rural roads, 1936-1950.

Table 5 gives comparisons of the estimated vehicle-mileage of travel by vehicles of different types on all main rural roads in 1936, the earliest year for which comprehensive travel and weight data are available; in 1941, the peak prewar year, 5 years after the beginning of the surveys; in 1946, 10 years after the beginning of the surveys; and in 1949 and 1950. The ratios of 1950 travel to that of the preceding years indicate that increases for trucks and truck combinations generally were greater than for passenger cars and busses, and that increases for truck combinations were greater than for single-unit trucks. In the 14 years from 1936 to 1950, passenger-car and bus travel combined increased 84 percent, travel by all trucks and combinations more than doubled, increasing 158 percent, and travel by truck combinations (considered separately) more than quadrupled, increasing 356 percent.

The lower portion of table 5 gives comparisons of the estimated vehicle-mileage of travel in 1936, 1949, and 1950 by privately operated trucks and truck combinations, and by those operated for-hire. Travel by for-hire vehicles increased somewhat more than travel by private vehicles, the 1950:1936 ratio being 3.32 in the case of for-hire vehicles and 2.36 in the case of private vehicles. Most of the increase in for-hire vehicle travel was by truck combinations, there being only a 67-percent increase in the for-hire vehicle-mileage by single-unit trucks compared to a 408-percent increase by combinations. In the case of the private vehicles, on the other hand, there were substantial increases in the vehicle-mileage by both types, the increase in the combinations, however, being much less than in the case of the for-hire vehicles.

Table 6.—*Comparison of the estimated percentage of trucks and truck combinations loaded, average carried load, and ton-miles carried on main rural roads in 1936, 1941, 1946, 1949, and 1950.*

Year	All trucks and truck combinations			Single-unit trucks			Truck combinations		
	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried
		Tons	*Millions*		*Tons*	*Millions*		*Tons*	*Millions*
1936	62.8	2.90	28,005	60.7	1.86	14,258	72.2	6.90	13,747
1941	66.7	3.64	58,737	65.4	2.29	28,487	71.6	8.23	30,250
1941:1936 ratio	*1.06*	*1.26*	*2.10*	*1.08*	*1.23*	*2.00*	*.99*	*1.19*	*2.20*
1946	51.7	4.84	60,892	46.4	2.31	19,101	66.2	9.70	41,791
1946:1941 ratio	*.78*	*1.33*	*1.04*	*.71*	*1.01*	*.67*	*.92*	*1.18*	*1.38*
1946:1936 ratio	*.82*	*1.67*	*2.17*	*.76*	*1.24*	*1.34*	*.92*	*1.41*	*3.04*
1949	51.6	5.11	89,100	46.1	2.29	25,639	65.7	10.19	63,461
1950	53.9	5.64	121,091	47.2	2.31	29,645	68.5	10.62	91,446
1950:1949 ratio	*1.04*	*1.10*	*1.36*	*1.02*	*1.01*	*1.16*	*1.04*	*1.04*	*1.44*
1950:1941 ratio	*.81*	*1.55*	*2.06*	*.72*	*1.01*	*1.04*	*.96*	*1.29*	*3.02*
1950:1936 ratio	*.86*	*1.94*	*4.32*	*.78*	*1.24*	*2.08*	*.95*	*1.54*	*6.65*
TRUCKS AND TRUCK COMBINATIONS IN PRIVATE OPERATION									
1936	60.3	2.20	16,094	59.8	1.71	11,180	65.5	6.37	4,914
1949	47.6	3.48	43,231	45.3	2.10	21,193	61.2	9.43	22,038
1950	49.1	3.69	52,509	46.2	2.07	23,370	64.5	9.96	29,139
1950:1949 ratio	*1.03*	*1.06*	*1.21*	*1.02*	*.99*	*1.10*	*1.05*	*1.06*	*1.32*
1950:1936 ratio	*.81*	*1.68*	*3.26*	*.77*	*1.21*	*2.09*	*.98*	*1.56*	*5.93*
TRUCKS AND TRUCK COMBINATIONS IN FOR-HIRE OPERATION									
1936	71.9	5.07	11,911	66.4	2.73	3,078	77.3	7.23	8,833
1949	65.1	9.16	45,869	55.1	3.97	4,446	68.7	10.65	41,423
1950	66.8	9.46	68,582	55.5	4.01	6,275	70.8	10.97	62,307
1950:1949 ratio	*1.03*	*1.03*	*1.50*	*1.01*	*1.01*	*1.41*	*1.03*	*1.03*	*1.50*
1950:1936 ratio	*.93*	*1.87*	*5.76*	*.84*	*1.47*	*2.04*	*.92*	*1.52*	*7.05*

Table 7.—*Percentage of vehicle-miles of travel, percentage loaded, average carried load, and percentage of total ton-miles carried by various types of trucks and truck combinations on main rural roads in 1950 compared to that in corresponding months of 1949*

Vehicle type	Percentage of vehicle-miles of travel		Percentage loaded		Average carried load		Percentage of ton-miles carried	
	1950	1949	1950	1949	1950	1949	1950	1949
					Tons	*Tons*		
Single-unit trucks:								
Panel and pick-up	31.35	31.55	37.4	35.9	0.69	0.64	2.65	2.75
Other 2-axle, 4-tire	2.79	3.46	52.4	49.4	.93	.78	.45	.50
Other 2-axle, 6-tire	32.63	35.34	55.9	54.5	3.20	3.17	19.06	23.15
3-axle	1.68	1.58	58.3	54.8	7.23	7.23	2.32	2.38
All single-unit trucks	68.45	71.93	47.2	46.1	2.31	2.29	24.48	28.78
Truck combinations:								
Truck-tractor and semitrailer	29.43	26.57	68.9	65.8	10.32	9.95	68.87	65.91
Truck and trailer	2.12	1.50	62.3	63.4	15.32	14.69	6.65	5.31
All truck combinations	31.55	28.07	68.5	65.7	10.62	10.19	75.52	71.22
All trucks and combinations	100.00	100.00	53.9	51.6	5.64	5.11	100.00	100.00
TRUCKS AND TRUCK COMBINATIONS IN PRIVATE OPERATION								
Single-unit trucks:								
Panel and pick-up	42.38	40.36	37.0	35.7	0.69	0.63	5.96	5.45
Other 2-axle, 4-tire	3.70	4.38	51.7	49.4	.87	.74	.92	.96
Other 2-axle, 6-tire	36.72	39.18	55.8	54.4	3.03	3.06	34.23	39.28
3-axle	1.54	1.45	57.1	53.4	6.99	7.11	3.40	3.33
All single-unit trucks	84.34	85.37	46.2	45.3	2.07	2.10	44.51	49.02
Truck combinations:								
Truck-tractor and semitrailer	14.78	13.85	64.3	61.1	9.88	9.39	51.80	47.98
Truck and trailer	.88	.78	67.8	63.6	11.21	10.08	3.69	3.00
All truck combinations	15.66	14.63	64.5	61.2	9.96	9.43	55.49	50.98
All trucks and combinations	100.00	100.00	49.1	47.6	3.69	3.48	100.00	100.00
TRUCKS AND TRUCK COMBINATIONS IN FOR-HIRE OPERATION]								
Single-unit trucks:								
Panel and pick-up	1.91	1.68	58.8	48.1	0.65	1.42	0.11	0.19
Other 2-axle, 4-tire	.37	.33	70.3	47.8	2.12	2.76	.09	.07
Other 2-axle, 6-tire	21.68	22.36	54.5	55.4	3.99	3.82	7.45	7.94
3-axle	2.02	2.03	60.8	58.1	7.68	7.52	1.50	1.49
All single-unit trucks	25.98	26.40	55.5	55.1	4.01	3.97	9.15	9.69
Truck combinations:								
Truck-tractor and semitrailer	68.59	69.63	71.6	69.0	10.55	10.27	81.93	82.82
Truck and trailer	5.43	3.97	59.9	63.2	17.34	17.75	8.92	7.49
All truck combinations	74.02	73.60	70.8	68.7	10.97	10.65	90.85	90.31
All trucks and combinations	100.00	100.00	66.8	65.1	9.46	9.16	100.00	100.00

Volume of Highway Freight

Figure 5 gives a comparison of the average load carried by single-unit trucks and truck combinations, separately and combined, in the 15 years that the planning surveys have been operating. The general trend of load weights was upward throughout the period. The slight decline in the weights of loads carried by single-unit trucks since 1945 has been more than offset by the increased use of combinations and the increased weights of loads carried by vehicles of this type.

Figure 6 shows, for each year from 1936 through 1950, the ton-mileage of freight carried by trucks and truck combinations on main rural roads. The chart demonstrates clearly that truck combinations are transporting each year a larger proportion of the total amount of highway freight. In 1936 the truck combinations hauled slightly less ton-mileage than the single-unit trucks, while in 1950 they hauled more than triple the amount transported by the larger number of lighter vehicles. The rapid rate of annual increase in total freight carried which took place in 1946 and 1947 was reduced somewhat in 1948 and 1949 to a rate of increase more nearly comparable with that of prewar years. In 1950, however, defense preparations appear to have been the cause of a rather startling increase in freight ton-mileage, somewhat similar to the rapid increase that occurred in 1941.

In table 6 are shown comparisons of the percentage of vehicles carrying loads, the average carried load, and the ton-mileage carried for single-unit trucks and truck combinations, separately and combined, in 1950 with corresponding items for other years, as in table 5. The trend from 1936 to 1950 of average weight carried, shown graphically in figure 5, and that of the ton-mileage transported during the same period, shown in figure 6, have already been discussed.

The percentage of trucks and truck combinations carrying loads increased noticeably from 1949 to 1950 in all regions except the West North Central region where a slight decrease of this factor was found. In the country as a whole, the percentage loaded increased from 51.6 percent in 1949 to 53.9 percent in 1950, an important factor in the striking increase in ton-mileage. Both for single-unit trucks and for truck combinations, the percentage loaded was higher in 1950 than in 1949, and, in the case of truck combinations, was higher than in any year since 1945. However, the loaded proportion was considerably less for each of the two vehicle types than in the prewar surveys.

The lower portion of table 6 shows comparisons of the percentage loaded, average carried load, and ton-mileage for single-unit trucks, truck combinations, and the two types of vehicles combined, when operated as private and as for-hire vehicles. A considerably larger percentage of the for-hire vehicles are loaded and the loads carried by these vehicles are much heavier than in the case of the privately operated vehicles. Single-unit trucks transport an important part of the freight moved in privately operated vehicles, but only a minor part of the freight moved in for-hire vehicles.

The first part of table 7 gives a detailed comparison of the percentage of vehicle-miles of travel, percentage of vehicles loaded, average carried load, and percentage of total ton-miles of freight carried by the various types of trucks and truck combinations traveling on main rural roads in 1949 and 1950. Many interesting comparisons can be made from this table showing the relative importance from a freight-carrying standpoint of different portions of the traffic stream. In 1950, for instance, while panel and pick-up trucks traveled more than 31 percent of the vehicle-mileage, they accounted for less than 3 percent of the ton-mileage. The truck-tractor and semitrailers, on the other hand, traveled about 29 percent of the vehicle-mileage but carried almost 69 percent of the ton-mileage.

From the columns in table 7 showing the percentage loaded, by types, it can be observed that the percentage of vehicles carrying loads tends to increase directly as the size of the vehicle type, extending from light panel and pick-up trucks that are loaded 37 percent of the time to the heavy combinations that are loaded about 69 percent of the time.

The lower portion of table 7 shows the same information separately for private and for-hire trucks. A comparison of vehicle-mileage percentage with ton-mileage percentage, by operating classes, shows that single-unit trucks, privately operated, traveled over 84 percent of the vehicle-mileage while transporting only about 44 percent of the freight moved in privately operated vehicles. At the same time, for-hire single-unit trucks traveled about 26 percent of the total for-hire vehicle-mileage and carried only about 9 percent of the total ton-mileage moved by the for-hire vehicles. The heavy vehicle combinations, privately operated, traveled about 16 percent of the total mileage and carried over 55 percent of the freight moved by privately operated vehicles, while the for-hire combinations traveled slightly more than 74 percent of the total vehicle-mileage of all for-hire vehicles and carried almost 91 percent of the freight transported by all vehicles in this class.

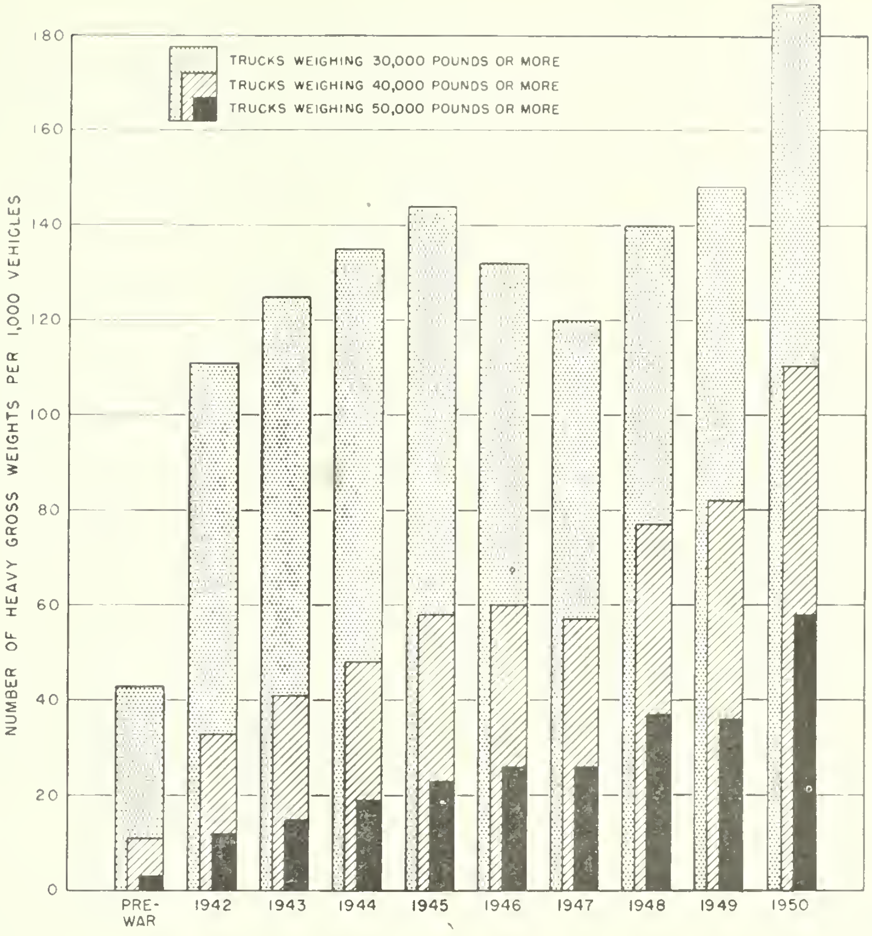

Figure 7.—Number of heavy gross weights per 1,000 trucks and truck combinations (empties included) in the summers of 1942-50 and a prewar year.

Gross Weights Increase Sharply

Figure 7 shows by years, from the prewar years (generally 1936 or 1937) to 1950, for the United States as a whole, the frequency of gross weights of 30,000 pounds or more, of 40,000 pounds or more, and 50,000 pounds or more. The chart shows strikingly how the frequency of heavy loads soared upward in 1950, reaching amounts for the various weights considerably above any previous levels. For instance, the frequency of the loads of 30,000 pounds or more was 26 percent higher than in 1949 and almost 30 percent higher than in 1945, the previous year of highest frequency of such loads. The increase in loads of 50,000 pounds or more was even more startling, the frequency being 61 percent above the 1949 figure and 152 percent above the 1945 figure. These heavy loads were over 19 times as frequent in 1950 as in the prewar year, loads of 40,000 pounds or more were 10 times as frequent, and those of 30,000 pounds or more were over 4 times as frequent as in the 1936–37 period.

The 1950 gross-weight frequency data by vehicle type and region are presented in table 8. No panels, pick-ups, or other two-axle, four-tire, single-unit trucks were found in the survey weighing as much as 30,000 pounds, so there is no entry for these vehicles in the table, though they are included in the total number of vehicles weighed in computing the frequencies for all trucks and combinations. Heavy gross weights are much more frequent in the Pacific region than in other parts of the country. In this region 176 of each 1,000 trucks and truck combinations on the main rural highways in 1950, empties included, weighed 50,000 pounds or more and 289 of each 1,000 weighed 30,000 pounds or more. In the East North Central region, 251 of each 1,000 trucks and truck combinations weighed 30,000 pounds or more—almost as many as in the Pacific region—but only 78 of each 1,000 vehicles weighed 50,000 pounds or more, a frequency less than half of that in the Pacific region for this heavy class of vehicle. The lowest frequency of heavy gross loads was found in the East South Central region where only 7 of each 1,000 weighed 50,000 pounds or more and only 102 of each 1,000 weighed 30,000 pounds or more.

As was pointed out in the discussion of figure 7, the frequencies of heavy gross loads have increased sharply in the Nation as a whole. This increase is not limited to any certain area but is distributed throughout the entire country. Comparing the frequencies of gross weights of 30,000 pounds or more, 40,000 pounds or more, and 50,000 pounds or more found in the 1950 surveys with such frequencies found in 1949, increases are found, without exception, in every region. For instance, in the East South Central region, where heavy gross loads are somewhat infrequent, the fre-

Table 8.—*Heavy gross weights per 1,000 loaded and empty trucks and truck combinations on main rural roads, summer of 1950*

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
NUMBER PER 1,000 WEIGHING 30,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	16	27	1	14	1	1	(1)	1	1	3	(1)	2	5
3-axle	280	284	327	305	303	236	286	264	283	407	234	281	289
Average	15	22	9	15	10	4	5	3	6	12	16	14	10
Truck combinations:													
Truck-tractor and semitrailer	521	606	540	566	575	473	571	522	549	600	688	656	568
Truck and trailer	0	(2)	(2)	(2)	585	0	172	306	491	743	710	717	622
Average	518	606	539	566	576	473	554	516	546	622	695	672	572
Average, all trucks and combinations	137	221	177	189	251	102	142	146	170	160	289	233	187
Comparative average, 1949	117	191	130	153	208	87	139	107	144	118	176	147	148
NUMBER PER 1,000 WEIGHING 40,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	1	2	0	1	0	0	0	0	0	1	0	1	(1)
3-axle	104	137	59	93	13	4	21	97	24	117	21	47	52
Average	3	4	2	3	(1)	(1)	(1)	1	(1)	4	1	3	2
Truck combinations:													
Truck-tractor and semitrailer	315	387	298	337	314	215	337	283	299	410	547	498	336
Truck and trailer	0	(2)	(2)	(2)	505	0	142	225	418	505	486	490	459
Average	313	388	297	337	329	215	328	281	304	425	529	496	345
Average, all trucks and combinations	78	135	95	109	140	45	82	79	95	106	214	167	110
Comparative average, 1949	66	120	71	90	105	36	77	54	73	75	121	97	82
NUMBER PER 1,000 WEIGHING 50,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 6-tire	0	0	0	0	0	0	0	0	0	1	0	(1)	(1)
3-axle	9	45	0	18	13	0	6	10	9	20	4	8	12
Average	(1)	1	0	(1)	(1)	0	(1)	(1)	(1)	1	(1)	1	(1)
Truck combinations:													
Truck-tractor and semitrailer	99	181	86	128	158	35	176	119	137	287	443	387	165
Truck and trailer	0	(2)	(2)	(2)	476	0	121	225	393	439	424	427	412
Average	98	183	87	129	183	35	174	122	150	311	437	397	182
Average, all trucks and combinations	24	63	28	41	78	7	44	34	47	76	176	133	58
Comparative average, 1949	15	52	21	33	48	6	32	18	29	51	99	75	36

[1] Less than 5 per 10,000.

[2] Data omitted because of insufficient sample.

quencies of loads of 40,000 pounds or more increased from 36 in 1949 to 45 in 1950; in the Pacific region the loads of 50,000 pounds or more increased from 99 in 1949 to 176 in 1950 for each 1,000 vehicles. The general prevalence of the heavier loads on the highways of all sections of the country gives a partial explanation of the large increase found in the ton-mileage of freight carried in 1950 compared to that carried in 1949.

Frequency of Heavy Axle Loads

Figure 8 shows the frequency of axle loads of 18,000 pounds or more, 20,000 pounds or more, and of 22,000 pounds or more for the prewar years (1936–37) and by years from 1942 to 1950. The frequency of these heavy axle loads increased year by year from the prewar period through 1948. The frequencies for 1949 were slightly lower than those found in 1948 yet they were higher than in any other previous year. The frequencies for 1950 are higher than those found in 1949, and the frequency of axles weighing 18,000 pounds

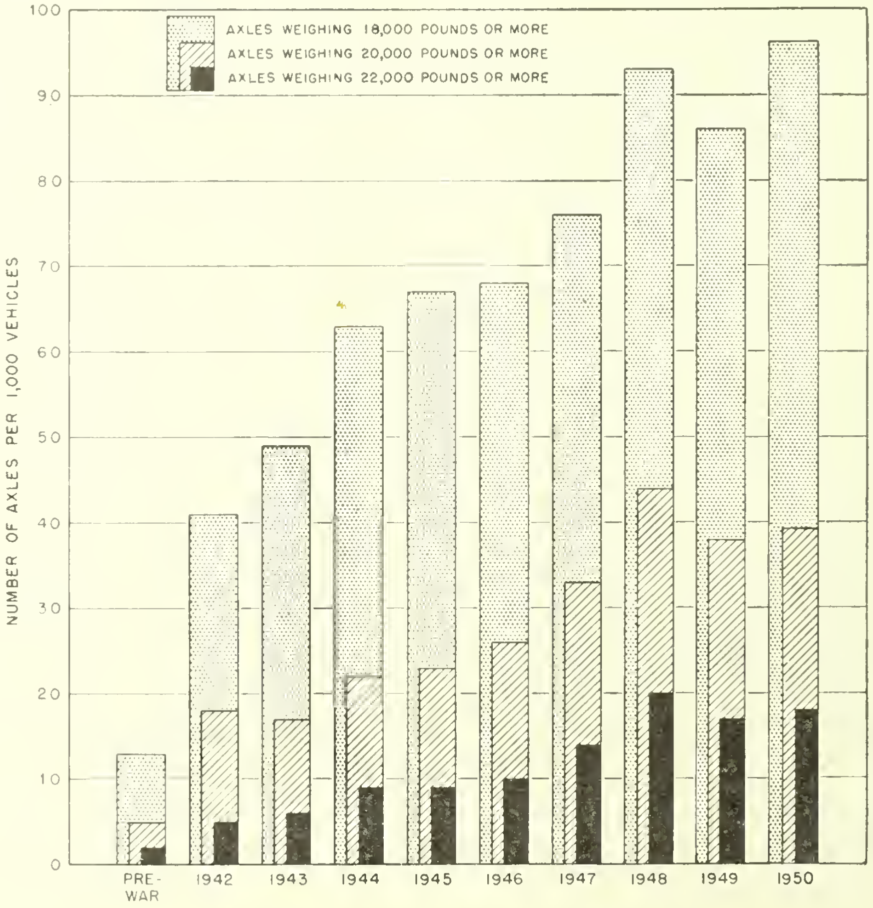

Figure 8.—Number of heavy axle loads per 1,000 trucks and truck combinations (empties included) in the summer of 1942-50 and a prewar year.

Table 9.—*Heavy axle loads per 1,000 loaded and empty trucks and truck combinations on main rural roads, summer of 1950*

Vehicle type	Eastern regions				Central regions					Western regions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Mountain	Pacific	Average	
NUMBER PER 1,000 WEIGHING 18,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 4-tire	0	9	0	4	8	(1)	0	0	2	0	0	0	3
2-axle, 6-tire	46	74	22	47	15	35	9	17	18	25	32	29	29
3-axle	107	179	63	111	39	43	20	78	41	154	26	61	68
Average	28	46	12	27	9	18	5	7	9	13	15	14	16
Truck combinations:													
Truck-tractor and semitrailer	484	524	289	407	204	236	165	227	206	316	177	227	276
Truck and trailer	0	345	0	169	403	0	160	(2)	323	212	82	107	193
Average	480	523	288	406	219	236	165	222	212	299	148	196	271
Average, all trucks and combinations	137	208	100	147	98	63	45	67	72	83	69	75	96
Comparative average, 1949	124	195	99	140	89	50	50	51	63	57	37	48	86
NUMBER PER 1,000 WEIGHING 20,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 4-tire	0	5	0	2	0	(1)	0	0	(1)	0	0	0	1
2-axle, 6-tire	31	47	6	27	2	10	3	7	5	6	7	6	12
3-axle	43	73	11	38	26	4	12	0	16	72	7	25	25
Average	18	28	3	15	2	5	1	3	3	4	4	4	7
Truck combinations:													
Truck-tractor and semitrailer	286	333	115	223	50	70	44	77	58	144	46	81	116
Truck and trailer	0	0	0	0	40	0	37	0	35	74	10	22	27
Average	284	331	114	222	49	70	44	75	56	133	35	66	110
Average, all trucks and combinations	82	131	38	80	22	19	12	23	19	35	16	24	39
Comparative average, 1949	73	118	46	78	27	18	12	18	20	26	6	16	38
NUMBER PER 1,000 WEIGHING 22,000 POUNDS OR MORE													
Single-unit trucks:													
2-axle, 4-tire	0	5	0	2	0	(1)	0	0	(1)	0	0	0	1
2-axle, 6-tire	19	27	2	15	1	3	1	2	1	2	2	2	6
3-axle	3	51	1	20	26	0	6	0	14	19	0	5	13
Average	11	17	1	8	1	1	1	1	1	1	1	1	3
Truck combinations:													
Truck-tractor and semitrailer	129	204	38	117	15	18	10	31	18	67	7	29	53
Truck and trailer	0	0	0	0	12	0	34	0	14	43	1	10	11
Average	128	203	38	116	15	18	11	30	18	64	5	24	50
Average, all trucks and combinations	39	80	13	42	7	5	3	9	6	16	3	9	18
Comparative average, 1949	33	65	18	39	9	5	3	6	6	11	2	6	17

(1) Less than 5 per 10,000.

(2) Data omitted because of insufficient sample.

or more is higher than in 1948, the previous high figure for that weight. The frequencies of axle loads weighing 20,000 pounds or more and those weighing 22,000 pounds or more, however, are lower in 1950 than in 1948. Altogether, the leveling off in the frequency of the heavier axle loads may possibly indicate that, although gross loads have increased sharply, more attention is being given to proper load distribution and that there is better observance of the axle-load restrictions.

Table 9 gives data concerning the number of heavy axle loads per 1,000 loaded and empty trucks and truck combinations of various types on the main rural roads by regions in 1950. Since no panel or pick-up trucks were found with axles weighing 18,000 pounds or more, there is no entry for these in the table though they are included in figuring the frequencies for all trucks and truck combinations.

Though the greatest frequency of heavy gross weights is in the Pacific region, as was shown in table 8, the lowest frequency of heavy axle loads is shared by that region with the West North Central region. In each of these two regions only three axles of 22,000 pounds or more were found in 1950 for each 1,000 vehicles weighed. By far the greatest frequency of heavy axle loads was in the Middle Atlantic region and the next greatest in New England. In these two regions the relatively high frequency is attributable mainly to the large number of two-axle truck-tractors pulling one-axle or two-axle semitrailers. The relative infrequency of heavy axles in the Pacific region, in the presence of a large proportion of heavy gross loads, indicates a better distribution of the loads over a larger number of axles.

Although the frequency of heavy gross loads has increased considerably and in all regions, as stated in connection with discussion of table 8, the trend in frequency of heavy axle loads followed an entirely different pattern. For the country as a whole, this was pointed out in the discussion of figure 8. The trend in frequency of heavy axle loads in the regions, likewise, is different from that of the gross loads. This is demonstrated by comparing the frequencies of heavy axle loads in 1950 with those in 1949 as shown for each weight class in table 9 and noting that the frequency of heavy axle loads in the different categories decreased in a number of cases, whereas table 8 shows that the frequency of heavy gross loads increased in all regions.

Loads Above Legal Limits

Table 10 shows the number of trucks and truck combinations of each type, per 1,000 such vehicles counted, empties included, that exceeded the legal axle, axle-group, or gross-weight limits in effect in the individual States in the summer of 1950, and the number per 1,000 that exceeded these limits by various percentages. Comparative figures are given at the bottom of the table, for the Nation as a whole, for 1948 and 1949.

Loads in excess of State law were most frequent in the East South Central region where a decided increase generally was found in the number of overloaded three-axle single-unit trucks and truck-tractor and semitrailer combinations. In this region, in 1949, 66 three-axle single-unit trucks of each 1,000 loaded and empty vehicles weighed exceeded one or more of the State weight limits; in 1950, 126 such vehicles exceeded these limits. In the same region 162 truck combinations per 1,000 such vehicles weighed in 1949 exceeded the legal limits while 437 exceeded these limits in 1950. After the East South Central region, where, of all loaded and empty trucks and truck combinations weighed in 1950, 115

Table 10.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle, axle-group, or gross-weight legal limits in effect in the States by various percentages (maximum) of overload summer of 1950*

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	15	10	6	2	1	(1)
3-axle	58	35	23	8	8	....
Average, single-unit trucks	10	6	4	1	1	(1)
Truck-tractor and semitrailer	113	74	45	19	6	2
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	112	74	45	19	6	2
Average, all trucks and combinations	35	22	14	5	2	(1)
Middle Atlantic:						
2-axle, 4-tire	(2)	....	....	....	....	....
2-axle, 6-tire	30	24	17	8	3	(1)
3-axle	119	116	94	31	11	11
Average, single-unit trucks	20	16	12	5	2	(1)
Truck-tractor and semitrailer	181	137	102	51	20	4
Truck and trailer	330	112	112	112	51	....
Average, truck combinations	182	137	102	51	20	4
Average, all trucks and combinations	75	57	43	21	8	1
South Atlantic:						
2-axle, 4-tire	(2)	(2)	....	....	....	....
2-axle, 6-tire	9	5	3	1	(1)	....
3-axle	39	20	14	4	1	1
Average, single-unit trucks	5	3	2	1	(1)	(1)
Truck-tractor and semitrailer	132	75	45	16	5	1
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	132	75	45	16	5	1
Average, all trucks and combinations	45	26	16	6	2	(1)
East North Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	9	4	1	1	(1)	
3-axle	72	72	26	13	13	13
Average, single-unit trucks	7	4	1	1	(1)	(1)
Truck-tractor and semitrailer	155	96	52	14	5	(1)
Truck and trailer	229	148	69	15	....	....
Average, truck combinations	161	100	53	14	5	(1)
Average, all trucks and combinations	72	45	23	7	2	(1)
East South Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	57	40	25	11	2	....
3-axle	126	49	19	....	....	....
Average, single-unit trucks	30	20	13	5	1	....
Truck-tractor and semitrailer	437	280	157	51	19	3
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	437	280	157	51	19	3
Average, all trucks and combinations	115	74	43	15	5	1
West North Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	9	4	3	1	(1)	....
3-axle	50	35	27	14	6	....
Average, single-unit trucks	5	3	2	1	(1)	....
Truck-tractor and semitrailer	188	115	69	25	12	2
Truck and trailer	77	72	58	37	34	34
Average, truck combinations	183	113	69	25	13	3
Average, all trucks and combinations	50	31	19	7	3	1
West South Central:						
2-axle, 4-tire		....	....	....	....	....
2-axle, 6-tire	16	12	8	2	1	....
3-axle	66	55	47	17	....	....
Average, single-unit trucks	7	5	4	1	(1)	....
Truck-tractor and semitrailer	213	139	90	43	21	3
Truck and trailer	(2)	(2)	....	....	....	....
Average, truck combinations	208	136	88	42	20	3
Average, all trucks and combinations	63	41	27	12	6	1
Mountain:						
2-axle, 4-tire	....	....	....	....	....	
2-axle, 6-tire	30	17	10	6	2	1
3-axle	137	121	114	34	29	5
Average, single-unit trucks	15	9	7	3	2	(1)
Truck-tractor and semitrailer	224	165	122	65	35	4
Truck and trailer	246	170	112	41	24	9
Average, truck combinations	227	166	120	61	33	5
Average, all trucks and combinations	67	47	34	17	10	1
Pacific:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	14	12	9	2	(1)	
3-axle	33	22	11	4	....	....
Average, single-unit trucks	8	7	5	1	(1)	....
Truck-tractor and semitrailer	162	97	54	22	8	1
Truck and trailer	257	157	115	83	51	1
Average, truck combinations	191	115	72	40	21	1
Average, all trucks and combinations	82	50	32	17	8	(1)
United States average:						
2-axle, 4-tire	2	(1)		....		....
2-axle, 6-tire	20	14	9	4	1	(1)
3-axle	70	55	38	13	7	4
Average, single-unit trucks	11	8	5	2	1	(1)
Truck-tractor and semitrailer	187	121	75	30	12	2
Truck and trailer	224	143	92	52	30	4
Average, truck combinations	189	122	76	31	13	2
Average, all trucks and combinations	67	44	27	11	5	1
Comparative average, 1949	51	35	23	10	4	1
Comparative average, 1948	55	38	26	12	6	1

[1] Less than 5 per 10,000. [2] Data omitted because of insufficient sample.

Table 11.—*Number of axles, per 1,000 loaded and empty trucks and truck combinations, that exceeded the permissible axle-load limit of 18,000 pounds recommended by the A.A.S.H.O. by various percentages of overload in the summer of 1950*

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	45	37	31	19	11	3
3-axle	97	56	49	14	....	....
Average, single-unit trucks	27	22	18	11	6	2
Truck-tractor and semitrailer	477	394	306	145	62	8
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	474	391	304	144	62	8
Average, all trucks and combinations	135	111	87	43	19	3
Middle Atlantic:						
2-axle, 4-tire	(1)	(1)	(1)	(1)	....	....
2-axle, 6-tire	69	56	47	28	15	7
3-axle	173	125	79	51	51	11
Average, single-unit trucks	43	35	29	17	10	4
Truck-tractor and semitrailer	504	422	345	224	132	35
Truck and trailer	319	82	....	....	....	....
Average, truck combinations	503	420	343	223	131	35
Average, all trucks and combinations	199	166	136	87	51	15
South Atlantic:						
2-axle, 4-tire	(1)	(1)	....	....	....	....
2-axle, 6-tire	14	8	6	2	(2)	....
3-axle	49	36	11	4	....	....
Average, single-unit trucks	8	5	3	1	(2)	....
Truck-tractor and semitrailer	261	184	123	47	16	2
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	260	183	123	47	16	2
Average, all trucks and combinations	88	61	41	16	5	1
East North Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	9	4	2	1	(2)	(2)
3-axle	26	26	26	26	13	....
Average, single-unit trucks	5	3	2	1	(2)	(2)
Truck-trailer and semitrailer	171	102	61	18	5	(2)
Truck and trailer	289	120	58	11	8	....
Average, truck combinations	180	103	61	18	5	(2)
Average, all trucks and combinations	79	46	27	6	2	(2)
East South Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	57	40	25	11	2	....
3-axle	112	39	39	....	....	....
Average, single-unit trucks	30	20	13	5	1	....
Truck-tractor and semitrailer	499	295	153	43	17	2
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	499	295	153	43	17	2
Average, all trucks and combinations	128	78	42	13	4	(2)
West North Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	9	4	3	1	(2)	....
3-axle	23	12	12	6	....	....
Average, single-unit trucks	5	2	2	1	(2)	....
Truck-tractor and semitrailer	155	82	48	12	4	(2)
Truck and trailer	102	52	40	....	....	....
Average, truck combinations	155	82	48	12	4	(2)
Average, all trucks and combinations	36	19	12	3	1	(2)
West South Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	16	12	8	3	1	....
3-axle	49	10	....	....	....	....
Average, single-unit trucks	7	5	3	1	(2)	....
Truck-tractor and semitrailer	214	128	82	35	16	2
Truck and trailer	(1)	(1)	....	....	....	....
Average, truck combinations	209	125	80	34	16	2
Average, all trucks and combinations	63	38	24	10	4	1
Mountain:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	30	17	10	6	2	1
3-axle	151	127	117	19	19	10
Average, single-unit trucks	15	10	7	3	1	1
Truck-tractor and semitrailer	225	146	89	37	18	1
Truck and trailer	177	101	51	12	12	3
Average, truck combinations	217	139	83	33	17	1
Average, all trucks and combinations	64	41	25	10	5	1
Pacific:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	14	12	9	2	(2)	....
3-axle	23	14	13	....	....	....
Average, single-unit trucks	8	6	5	1	(2)	....
Truck-tractor and semitrailer	134	76	43	10	4	1
Truck and trailer	129	45	15	1	(2)	(2)
Average, truck combinations	132	67	35	7	3	1
Average, all trucks and combinations	58	31	17	3	1	(2)
United States average:						
2-axle, 4-tire	3	1	1	1	....	....
2-axle, 6-tire	28	20	15	8	3	1
3-axle	65	44	33	14	10	2
Average, single-unit trucks	15	11	8	4	2	1
Truck-tractor and semitrailer	269	185	127	60	29	6
Truck and trailer	174	72	32	5	4	(2)
Average, truck combinations	263	177	121	56	27	6
Average, all trucks and combinations	93	63	44	20	10	3
Comparative average, 1949	75	54	37	17	8	1
Comparative average, 1948	85	63	45	23	11	2

[1] Data omitted because of insufficient sample. [2] Less than 5 per 10,000.

Table 12.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded the permissible axle-group loads recommended by the A.A.S.H.O. by various percentages of overload in the summer of 1950*

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	(1)	(1)	(1)	....	....	....
3-axle	80	47	27	8	8	....
Average, single-unit trucks	2	1	1	(1)	(1)	....
Truck-tractor and semitrailer	74	44	25	12	7	1
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	74	44	25	12	7	1
Average, all trucks and combinations	19	11	7	3	2	(1)
Middle Atlantic:						
2-axle, 4-tire	(2)	(2)	(2)	....	....	....
2-axle, 6-tire	1	(1)	(1)	(1)	(1)	(1)
3-axle	130	116	108	74	25	11
Average, single-unit trucks	4	3	3	2	1	(1)
Truck-tractor and semitrailer	156	127	95	50	26	10
Truck and trailer	390	299	278	112	103	....
Average, truck combinations	157	128	96	50	26	10
Average, all trucks and combinations	56	45	85	18	9	3
South Atlantic:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	....	....	....	....	....	....
3-axle	73	30	21	6	....	....
Average, single-unit trucks	2	1	1	(1)	....	....
Truck-tractor and semitrailer	59	40	26	12	5	2
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	59	40	26	12	5	2
Average, all trucks and combinations	20	13	9	4	2	1
East North Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	....	....	....	....	....	....
3-axle	33	26	26	13	13	13
Average, single-unit trucks	1	1	1	(1)	(1)	(1)
Truck-tractor and semitrailer	137	98	57	15	3	....
Truck and trailer	447	406	366	195	77	24
Average, truck combinations	161	122	81	29	9	2
Average, all trucks and combinations	69	52	35	12	4	1
East South Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	....	....	....	....	....	....
3-axle	8	4	....	....	....	....
Average, single-unit trucks	(1)	(1)	....	....	....	....
Truck-tractor and semitrailer	36	22	12	4	1	....
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	36	22	12	4	1	....
Average, all trucks and combinations	8	5	3	1	(1)	....
West North Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	....	....	....	....	....	....
3-axle	27	21	14	6	6	....
Average, single-unit trucks	(1)	(1)	(1)	(1)	(1)	....
Truck-tractor and semitrailer	111	68	40	15	6	(1)
Truck and trailer	71	54	40	34	34	34
Average, truck combinations	109	67	40	16	7	1
Average, all trucks and combinations	27	17	10	4	2	(1)
West South Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	....	....	....	....	....	....
3-axle	47	47	37	....	....	....
Average, single-unit trucks	(1)	(1)	(1)	....	....	....
Truck-tractor and semitrailer	98	65	40	15	6	1
Truck and trailer	(2)	(2)	(2)	(2)	(2)	....
Average, truck combinations	96	63	39	15	6	1
Average, all trucks and combinations	27	18	11	4	2	(1)
Mountain:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	(1)	(1)	(1)	(1)	....	....
3-axle	118	83	58	25	10	5
Average, single-unit trucks	3	2	2	1	(1)	(1)
Truck-tractor and semitrailer	178	143	108	54	27	2
Truck and trailer	214	155	105	49	23	6
Average, truck combinations	184	145	108	53	26	3
Average, all trucks and combinations	47	37	28	14	6	1
Pacific:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	....	....	....	....	....	....
3-axle	27	19	6	4	....	....
Average, single-unit trucks	2	1	(1)	(1)	....	....
Truck-tractor and semitrailer	265	206	135	43	8	1
Truck and trailer	369	267	132	13	3	....
Average, truck combinations	296	224	134	34	6	1
Average, all trucks and combinations	120	91	54	14	2	(1)
United States average:						
2-axle, 4-tire	1	1	1	....	....	....
2-axle, 6-tire	(1)	(1)	(1)	(1)	(1)	(1)
3-axle	56	40	31	16	7	4
Average, single-unit trucks	1	1	1	(1)	(1)	(1)
Truck-tractor and semitrailer	123	89	58	23	9	2
Truck and trailer	337	268	185	71	30	10
Average, truck combinations	137	101	67	26	10	8
Average, all trucks and combinations	44	33	22	8	8	1
Comparative average, 1949	28	21	14	7	3	1
Comparative average, 1948	30	22	16	7	3	1

[1] Less than 5 per 10,000. [2] Data omitted because of insufficient sample.

Table 13.—*Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded any of the permissible load limits recommended by the A.A.S.H.O. by various percentages (maximum) of overload in the summer of 1950*

Region and type of vehicle	Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent
New England:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	45	37	31	19	11	3
3-axle	121	93	62	21	8	....
Average, single-unit trucks	28	23	19	11	6	2
Truck-tractor and semitrailer	310	268	214	112	52	7
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	308	266	213	111	52	7
Average, all trucks and combinations	95	82	66	35	17	3
Middle Atlantic:						
2-axle, 4-tire	(1)	(1)	(1)	(1)	....	....
2-axle, 6-tire	69	56	47	28	15	7
3-axle	134	119	108	74	40	11
Average, single-unit trucks	42	35	30	18	9	4
Truck-tractor and semitrailer	341	298	240	158	95	33
Truck and trailer	411	319	278	112	103	....
Average, truck combinations	341	298	240	158	95	33
Average, all trucks and combinations	143	124	101	66	38	14
South Atlantic:						
2-axle, 4-tire	(1)	(1)	....	....	....	....
2-axle, 6-tire	14	8	6	2	(2)	....
3-axle	90	41	26	10	....	....
Average, single-unit trucks	9	5	3	1	(2)	....
Truck-tractor and semitrailer	199	144	99	45	17	3
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	198	144	99	45	17	3
Average, all trucks and combinations	69	49	33	15	5	1
East North Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	9	4	2	1	(2)	(2)
3-axle	54	26	26	13	13	13
Average, single-unit trucks	6	3	2	1	(2)	(2)
Truck-tractor and semitrailer	219	150	92	27	8	(2)
Truck and trailer	449	409	367	196	84	24
Average, truck combinations	237	170	113	40	14	2
Average, all trucks and combinations	104	74	49	18	6	1
East South Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	57	40	25	11	2	....
3-axle	116	43	39	....	....	....
Average, single-unit trucks	30	20	13	5	1	....
Truck-tractor and semitrailer	395	252	136	41	15	2
Truck and trailer	....	....	....	....	....	....
Average, truck combinations	395	252	136	41	15	2
Average, all trucks and combinations	107	69	39	13	4	(2)
West North Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	9	4	3	1	(2)	....
3-axle	30	16	14	6	6	....
Average, single-unit trucks	5	2	2	1	(2)	....
Truck-tractor and semitrailer	179	108	63	20	8	1
Truck and trailer	75	62	44	37	34	34
Average, truck combinations	175	106	62	21	9	2
Average, all trucks and combinations	48	28	17	6	2	1
West South Central:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	16	12	8	3	1	....
3-axle	67	57	37	....	....	....
Average, single-unit trucks	7	5	3	1	(2)	....
Truck-tractor and semitrailer	204	136	87	36	18	2
Truck and trailer	(1)	(1)	(1)	(1)	(1)	....
Average, truck combinations	199	133	85	35	18	2
Average, all trucks and combinations	60	41	26	10	5	1
Mountain:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	30	17	10	6	2	1
3-axle	137	115	110	47	19	5
Average, single-unit trucks	15	9	7	3	1	(2)
Truck-tractor and semitrailer	240	187	142	71	35	4
Truck and trailer	290	210	124	52	32	9
Average, truck combinations	248	191	139	68	35	5
Average, all trucks and combinations	72	53	39	19	9	1
Pacific:						
2-axle, 4-tire	....	....	....	....	....	....
2-axle, 6-tire	14	12	9	2	(2)	....
3-axle	37	25	12	4	2	....
Average, single-unit trucks	9	7	5	1	(2)	....
Truck-tractor and semitrailer	293	230	150	48	11	1
Truck and trailer	396	277	138	13	3	(2)
Average, truck combinations	324	244	146	37	9	1
Average, all trucks and combinations	136	102	62	15	4	(2)
United States average:						
2-axle, 4-tire	3	2	2	1	....	....
2-axle, 6-tire	28	20	15	8	3	1
3-axle	77	50	41	18	10	4
Average, single-unit trucks	15	11	8	4	2	1
Truck-tractor and semitrailer	249	183	125	56	26	6
Truck and trailer	359	280	191	72	33	10
Average, truck combinations	256	190	129	57	26	6
Average, all trucks and combinations	91	68	46	21	10	3
Comparative average, 1949	68	53	38	19	10	2
Comparative average, 1948	73	56	42	23	11	3

[1] Data omitted because of insufficient sample. [2] Less than 5 per 10,000.

Table 14.—*Number of trucks and truck combinations per 1,000 loaded and empty vehicles, in private and in for-hire operation, that exceeded various load limits by various percentages of overload in the summer of 1950 (United States average)*

Type of vehicle	Private operation						For-hire operation					
	Number per 1,000 overloaded	Number per 1,000 overloaded more than—					Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent		5 percent	10 percent	20 percent	30 percent	50 percent
NUMBER OF TRUCKS AND TRUCK COMBINATIONS PER 1,000 EXCEEDING PERMISSIBLE AXLE, AXLE-GROUP, OR GROSS-WEIGHT LEGAL LIMITS OF THE SEVERAL STATES												
2-axle, 4-tire	1	(1)	....	....	....	....	(2)	....	....	....	....	....
2-axle, 6-tire	16	10	6	2	1	(1)	39	27	19	10	3	(1)
3-axle	71	57	41	10	6	5	63	52	33	16	7	3
Average, single-unit trucks	8	5	3	1	1	(1)	38	27	18	10	3	(1)
Truck-tractor and semitrailer	173	108	68	29	14	2	196	129	79	30	12	2
Truck and trailer	122	65	42	27	21	4	267	174	112	61	32	3
Average, truck combinations	170	106	67	29	14	2	201	132	81	32	13	2
Average, all trucks and combinations	33	21	13	5	3	(1)	159	105	65	26	10	1
Comparative average, 1949	26	18	13	5	3	1	131	89	56	23	11	2
NUMBER OF AXLES PER 1,000 TRUCKS AND TRUCK COMBINATIONS EXCEEDING THE 18,000-POUND LIMIT RECOMMENDED BY THE A.A.S.H.O.												
2-axle, 4-tire	2	(1)	....	....	....	....	(2)	(2)	(2)	(2)	....	....
2-axle, 6-tire	22	15	11	5	2	1	55	40	30	18	8	5
3-axle	57	43	35	14	9	1	79	46	29	14	12	4
Average, single-unit trucks	11	7	5	2	1	(1)	52	37	28	16	8	4
Truck-tractor and semitrailer	278	193	133	62	28	6	263	180	123	58	29	7
Truck and trailer	110	39	15	4	4	1	202	86	39	6	3	(1)
Average, truck combinations	269	184	126	59	27	6	259	173	117	54	27	6
Average, all trucks and combinations	51	35	24	11	3	1	205	138	94	44	22	5
Comparative average, 1949	39	28	20	8	3	1	195	141	99	46	21	5
NUMBER OF TRUCKS AND TRUCK COMBINATIONS PER 1,000 EXCEEDING THE MAXIMUM AXLE-GROUP LOADS RECOMMENDED BY THE A.A.S.H.O.												
2-axle, 4-tire	1	1	1	....	....	....	....	....	....	....	....	....
2-axle, 6-tire	(1)	(1)	(1)	(1)	....	....	(1)	(1)	(1)	(1)	(1)	(1)
3-axle	63	43	31	17	7	6	40	32	28	12	6	1
Average, single-unit trucks	1	1	1	(1)	(1)	(1)	3	2	2	1	(1)	(1)
Truck-tractor and semitrailer	101	71	44	18	8	2	136	101	66	25	9	2
Truck and trailer	175	118	85	22	14	4	405	330	229	93	36	11
Average, truck combinations	105	74	46	18	8	2	156	118	78	30	11	3
Average, all trucks and combinations	17	12	8	3	1	(1)	116	88	58	22	8	2
Comparative average, 1949	12	8	6	3	2	(1)	87	64	43	20	10	2
NUMBER OF TRUCKS AND TRUCK COMBINATIONS PER 1,000 EXCEEDING ANY OF THE MAXIMUM MOTOR-VEHICLE LOADS RECOMMENDED BY THE A.A.S.H.O.												
2-axle, 4-tire	2	1	1	....	....	....	(2)	(2)	(2)	(2)	....	....
2-axle, 6-tire	22	15	11	5	2	1	55	40	30	18	8	5
3-axle	77	52	41	19	9	5	71	45	40	16	11	3
Average, single-unit trucks	11	8	6	3	1	1	52	37	29	17	8	4
Truck-tractor and semitrailer	229	167	113	53	25	5	260	191	130	58	27	6
Truck and trailer	181	119	76	16	12	3	428	343	234	93	38	11
Average, truck combinations	226	164	111	51	24	5	272	202	138	61	28	6
Average, all trucks and combinations	45	32	22	11	5	2	215	159	110	50	23	5
Comparative average, 1949	34	26	19	9	4	1	184	143	103	52	25	6

[1] Less than 5 per 10,000. [2] Data omitted because of insufficient sample.

exceeded one or more of the State weight limits, the Pacific region had the second highest rate of overloads (82) and in descending order of rates of violation were the Middle Atlantic (75), the East North Central (72), the Mountain (67), the West South Central (63), the West North Central (50), the South Atlantic (45), and the New England region (35).

A comparison of the frequency of loads exceeding State limits in 1950, shown in table 10, with similar data collected in the previous year, indicates that the frequency of these illegal loads has increased in all regions except the South Atlantic, in which this frequency decreased from 53 to 45 per 1,000 vehicles weighed. In all other regions increases in the rate of weight violations were found although the increases did not extend to the larger percentages of violation. For instance, in the East North Central region 63 vehicles of each 1,000 weighed in 1949 exceeded one or more of the weight restrictions by some amount, while in 1950, 72 vehicles per 1,000 exceeded the restrictions. At the same time, of those weighed in 1949, 27 exceeded the limits by more than 10 percent, while in 1950, only 23 exceeded these limits by more than 10 percent.

No panel or pick-up truck was weighed that exceeded any of the State weight regulations and this classification is omitted from tables 10–14 although the number of such vehicles counted is included in the calculations.

Recommended Weight Limits

Uniform regulations concerning maximum allowable gross weights, axle weights, and axle-group weights have been adopted as a policy by the American Association of State Highway Officials and recommended to the State governments for adoption.[7] This policy recommends that no axle shall carry a load in excess of 18,000 pounds and no group of axles shall carry a load in excess of amounts specified in a table of permissible weights based on the distance between the extremes of any group of axles.

In table 11 is shown the number of axles per 1,000 vehicles of various types that exceeded the axle load limit of 18,000 pounds recommended by the A.A.S.H.O. and the number exceeding these limits by various percentages. This table emphasizes again the high frequency of heavy axle loads in the Middle Atlantic and New England regions. The number of axles per 1,000 vehicles weighing more than the A.A.S.H.O. recommended limits was 199 in the Middle Atlantic and 135 in the New England region, while only 58 such axles for each 1,000 vehicles were found in the Pacific region and 36 in the West North Central region. There were 87 axles per 1,000 vehicles in the Middle Atlantic region exceeding the 18,000-pound recommended limit by 20 percent or more, compared to only 3 each in the Pacific and West North Central regions.

Table 12 shows the number of vehicles of various types, per 1,000 vehicles, with an axle-group load in excess of the limits recommended by the A.A.S.H.O. and in excess of the limits by various percentages. As might be expected from the large increases of frequencies of heavy gross loads indicated in figure 7, the number of vehicles of various types per 1,000 weighed that exceeded the A.A.S.H.O. recommendations increased in 1950 over the similar rates in 1949. For the country as a whole, of each 1,000 loaded and empty trucks and truck combinations, 44 had axle groups in 1950 weighing in excess of the recommended limits, 8 of which exceeded the limits by more than 20 percent. In 1949, comparable figures indicated that 28 trucks and truck combinations of each 1,000 exceeded the axle-group recommendation, 7 of which exceeded the limits by more than 20 percent. Of each 1,000 combinations weighed, 137 had axle-group loads weighing more than the recommended limits, of which 26 exceeded the limits by more than 20 percent. The frequency of the excessive axle-group loads in 1950 was about 57 percent more than in 1949.

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[7] *Policy concerning maximum dimensions, weights, and speeds of motor vehicles to be operated over the highways of the United States,* adopted April 1, 1946, by the American Association of State Highway Officials; published by the Association in 1946.

It will be noted that a higher proportion of the vehicles have excessive axle-group loads in the Pacific region than elsewhere, whereas table 11 shows a comparatively low frequency of heavy axle loads for that region. This is because of the widespread use of multiple-axle vehicles in California and neighboring States.

As might be expected, many vehicles were so loaded that they exceeded more than one recommended weight limit, and some vehicles had more than one axle loaded in excess of the recommended limit. Counting each vehicle only once, regardless of the number of ways in which it exceeded any of the A.A.S.H.O. recommended limits, table 13 was prepared to show the number of vehicles per 1,000 of each type, both loaded and empty, that exceeded the limits by various percentages. Those vehicles which exceeded more than one provision of the recommended restrictions were tabulated only in the column showing the highest percentage excess of any item.

In the United States as a whole, 91 vehicles out of every 1,000 were overloaded to some degree and 21 out of every 1,000 exceeded some one of the recommended provisions by more than 20 percent. The frequency of vehicles exceeding the recommendations by any amount in 1950 was 34 percent more than in 1949, when 68 vehicles out of every 1,000 were overloaded to some degree. The frequency exceeding the recommendations by more than 20 percent in 1950 was 11 percent more than in 1949, when 19 vehicles out of every 1,000 vehicles exceeded some recommended limit to this extent.

State Limits Higher

In considering the data concerning the frequencies of axles or vehicles exceeding the State legal limits and the A.A.S.H.O. recommendations, especially the frequencies in the Middle Atlantic and New England regions, the fact should be recognized that higher limits generally are permissible under the State laws in these areas than are recommended by the Association. Axles exceeding the recommended limits by 25 percent may be within the legal limits of certain States, particularly in these two regions. Some States have no axle-group limits in their motor-vehicle restrictions, a fact that further complicates direct comparison of excess weights based on law and those based on the recommendations. Comparison of the frequency data for New England and the Middle Atlantic regions given in table 13 with those in table 10 shows that from one-third to one-half of the vehicles exceeding one or more of the Association recommendations actually exceeded a State legal limit. For the United States as a whole, nearly three-fourths of the vehicles exceeding one or more of the Association recommendations also exceeded a State legal limit.

Overloading of For-Hire Vehicles

The first part of table 14 shows separately the number of privately operated trucks and truck combinations and those operated for-hire, for each 1,000 such loaded and empty vehicles on main rural roads of the United States, that exceeded some State legal weight limit in 1950, and also comparative average figures for 1949. A comparison of the frequency of the excessively loaded vehicles in the two operation classifications shows, in striking manner, that type by type the for-hire vehicles generally are more frequently overloaded than are the privately operated ones. For instance, 8 of each 1,000 private single-unit trucks exceeded a State weight limit while 38 of each 1,000 for-hire trucks exceeded the same limits. Likewise, 170 of each 1,000 private truck combinations exceeded State weight limits, while 201 of each 1,000 for-hire combinations exceeded the same limits.

Of each 1,000 vehicles, the frequencies of all private and all for-hire trucks and truck combinations exceeding the State limits in 1950 were 33 and 159, respectively, while in the previous year the corresponding frequencies were 26 and 131. In both years, there were nearly five times as many excess loads among the for-hire vehicles as among the privately operated ones.

The following parts of table 14 show frequencies of private and for-hire trucks and truck combinations exceeding the A.A.S.H.O. recommended limits for axle loads, for maximum axle-group loads, or for any of the recommended maximum loads. These sections of the table show, in general, as did the first section, that the relation of the frequency of overload of privately operated and for-hire vehicles is approximately the same when based on A.A.S.H.O. recommendations as when based on State legal limits.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

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Factual Discussion of Motortruck Operation, Regulation, and Taxation. 30 cents.
Federal Legislation and Regulations Relating to Highway Construction. 40 cents.
Financing of Highways by Counties and Local Rural Governments, 1931–41. 45 cents.
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Highway Bond Calculations. 10 cents.
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Highway Statistics, 1946. 50 cents.
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ANNUAL REPORTS

(See also adjacent column)

Public Roads Administration Annual Reports:
1943. 1944. 1945.

MISCELLANEOUS PUBLICATIONS

Bibliography on Automobile Parking in the United States.
Bibliography on Highway Lighting.
Bibliography on Highway Safety.
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Express Highways in the United States: a Bibliography.
Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.
Title sheets for PUBLIC ROADS, volumes 24 and 25.

U.S. GOVERNMENT PRINTING OFFICE: 1951-951446

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF OCTOBER 31, 1951

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$11,992	$16,872	$9,011	316.6	$4,411	$2,050	71.8	$19,321	$9,682	397.2	$40,604	$20,743	785.6
Arizona	462	4,153	2,853	80.8	1,259	890	36.4	5,660	4,011	68.2	11,072	7,754	185.4
Arkansas	1,869	8,299	4,530	310.5	7,254	3,586	216.8	12,252	6,317	354.9	27,805	14,433	882.2
California	6,021	17,157	2,720	55.6	6,899	3,280	18.9	69,982	33,176	329.6	94,038	39,176	404.1
Colorado	2,087	3,559	1,992	87.5	2,140	1,114	44.1	14,572	7,917	293.5	20,271	11,023	425.1
Connecticut	3,251	3,264	1,682	11.3	1,018	508	.8	12,283	6,388	18.4	16,565	8,578	30.5
Delaware	2,033	3,497	1,096	16.7	490	248	13.5	4,608	2,237	28.6	8,595	3,581	58.8
Florida	3,793	9,385	4,889	223.3	8,987	4,610	152.5	17,284	8,621	325.7	35,656	18,120	701.5
Georgia	3,145	14,474	7,393	359.5	9,720	4,356	118.0	28,277	14,147	422.8	52,471	25,896	900.3
Idaho	3,067	11,056	6,908	349.5	639	397	19.2	4,486	2,939	105.7	16,181	10,244	474.4
Illinois	17,314	32,073	17,828	230.5	18,047	9,119	150.3	71,052	35,807	618.0	121,172	62,754	998.8
Indiana	10,573	29,843	15,038	161.4	9,519	4,602	64.2	17,786	9,048	212.3	57,148	28,688	437.9
Iowa	1,036	9,852	5,277	308.6	5,186	2,620	162.5	17,071	8,458	705.6	32,109	16,355	1,176.7
Kansas	5,466	8,741	4,233	1,055.1	2,555	1,286	325.3	16,447	8,190	565.5	27,743	13,709	1,945.9
Kentucky	917	17,154	8,973	264.6	7,039	3,592	182.0	11,774	5,872	149.5	35,967	18,437	596.1
Louisiana	2,279	17,377	8,338	80.8	7,571	3,503	77.9	19,948	10,111	167.9	44,896	21,952	326.6
Maine	1,559	6,896	3,811	44.1	3,170	1,553	25.7	5,246	2,670	49.9	15,312	8,034	119.7
Maryland	4,533	2,230	1,032	22.3	6,326	3,344	19.5	9,854	4,211	39.8	18,410	8,587	81.6
Massachusetts	485	10,456	4,373	24.6	10,660	4,977	10.1	58,005	28,695	52.1	79,121	38,045	86.8
Michigan	1,808	22,388	11,201	427.6	7,098	3,555	134.2	48,935	20,936	296.5	78,421	35,692	858.3
Minnesota	4,227	5,098	2,671	771.8	1,703	972	165.8	26,775	14,282	933.1	33,576	17,925	1,870.7
Mississippi	7,452	4,768	2,364	196.6	3,454	1,682	113.4	19,190	9,951	537.2	27,412	13,997	847.2
Missouri	8,172	22,277	11,754	653.8	9,654	4,742	190.6	37,281	19,565	682.3	69,212	36,061	1,526.7
Montana	4,128	13,782	8,404	392.7	3,030	1,806	32.6	13,478	7,965	224.7	30,290	18,175	650.0
Nebraska	5,368	16,612	8,602	622.6	4,632	2,333	68.6	16,230	7,998	547.2	37,474	18,933	1,238.4
Nevada	2,619	3,795	3,168	101.7	607	496	2.7	2,943	2,462	154.5	7,345	6,126	258.9
New Hampshire	1,834	2,940	1,654	16.4	681	339	4.2	4,448	2,210	34.7	8,069	4,203	55.3
New Jersey	1,961	9,133	4,447	19.0	11,082	5,427	9.4	18,558	8,943	27.3	38,773	18,817	55.7
New Mexico	1,426	3,833	2,453	149.9	659	433	12.3	11,935	7,631	213.6	16,427	10,517	375.8
New York	22,777	79,409	41,543	203.6	14,514	6,582	62.0	110,168	51,340	403.3	204,091	99,465	668.9
North Carolina	2,462	17,318	8,571	331.7	2,729	1,292	66.5	25,302	12,637	526.4	45,349	22,500	924.6
North Dakota	1,101	6,667	3,460	1,049.0	5,003	2,504	582.3	9,400	4,714	766.1	21,070	10,678	2,397.4
Ohio	10,267	28,217	13,621	217.9	10,860	5,348	56.5	68,653	34,807	267.7	107,730	53,776	542.1
Oklahoma	2,196	12,648	6,882	199.5	3,892	2,077	49.1	19,645	10,556	256.1	36,185	19,515	504.7
Oregon	652	1,927	1,139	18.6	1,318	691	20.8	15,820	8,782	200.9	19,065	10,612	240.3
Pennsylvania	3,253	24,803	12,395	32.0	20,248	9,864	62.4	57,716	28,644	180.2	102,767	50,903	274.6
Rhode Island	1,625	5,095	2,547	42.5	792	395	3.0	14,918	7,583	12.8	20,805	10,525	58.3
South Carolina	1,363	11,514	6,282	253.6	2,987	1,439	107.6	10,802	5,464	182.1	25,303	13,185	543.3
South Dakota	696	4,243	2,509	410.6	3,852	2,280	281.9	12,046	6,875	687.3	20,141	11,664	1,379.8
Tennessee	2,181	11,845	5,592	363.7	8,054	3,704	98.3	24,889	11,848	330.6	44,788	21,144	792.6
Texas	7,819	1,972	874	67.3	9,901	5,512	101.0	56,163	26,746	532.7	68,036	33,132	701.0
Utah	1,046	6,296	4,733	126.6	1,133	814	21.4	3,689	2,670	50.5	11,118	8,217	198.5
Vermont	493	4,168	2,294	44.1	1,056	527	12.1	4,930	2,447	29.9	10,154	5,268	86.1
Virginia	4,537	16,507	8,174	408.6	6,076	3,118	157.8	20,101	9,913	320.5	42,684	21,205	886.9
Washington	1,545	9,996	4,220	143.0	2,395	1,400	30.1	19,525	9,194	114.1	31,916	14,814	287.2
West Virginia	2,714	10,891	5,473	130.3	3,632	1,829	41.0	11,025	5,504	130.8	25,548	12,806	302.1
Wisconsin	5,451	18,983	10,443	330.9	6,943	3,380	102.3	20,832	10,344	441.8	46,758	24,167	875.0
Wyoming	328	1,204	877	25.4	1,329	867	27.9	6,754	4,369	199.1	9,287	6,113	252.4
Hawaii	832	7,837	3,387	11.1	843	412	3.8	8,373	3,066	23.8	17,053	6,865	38.7
District of Columbia	3,535	2,864	1,432	.7				3,953	2,169	2.8	6,817	3,601	3.5
Puerto Rico	3,029	10,336	4,820	46.8	1,385	622	2.9	10,019	4,594	36.2	21,740	10,036	85.9
TOTAL	200,779	625,704	319,963	11,812.9	264,432	132,077	4,334.0	1,150,404	574,706	14,252.0	2,040,540	1,026,746	30,398.9

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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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Evening peak traffic on the Shirley Highway

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U. S. DEPARTMENT OF COMMERCE
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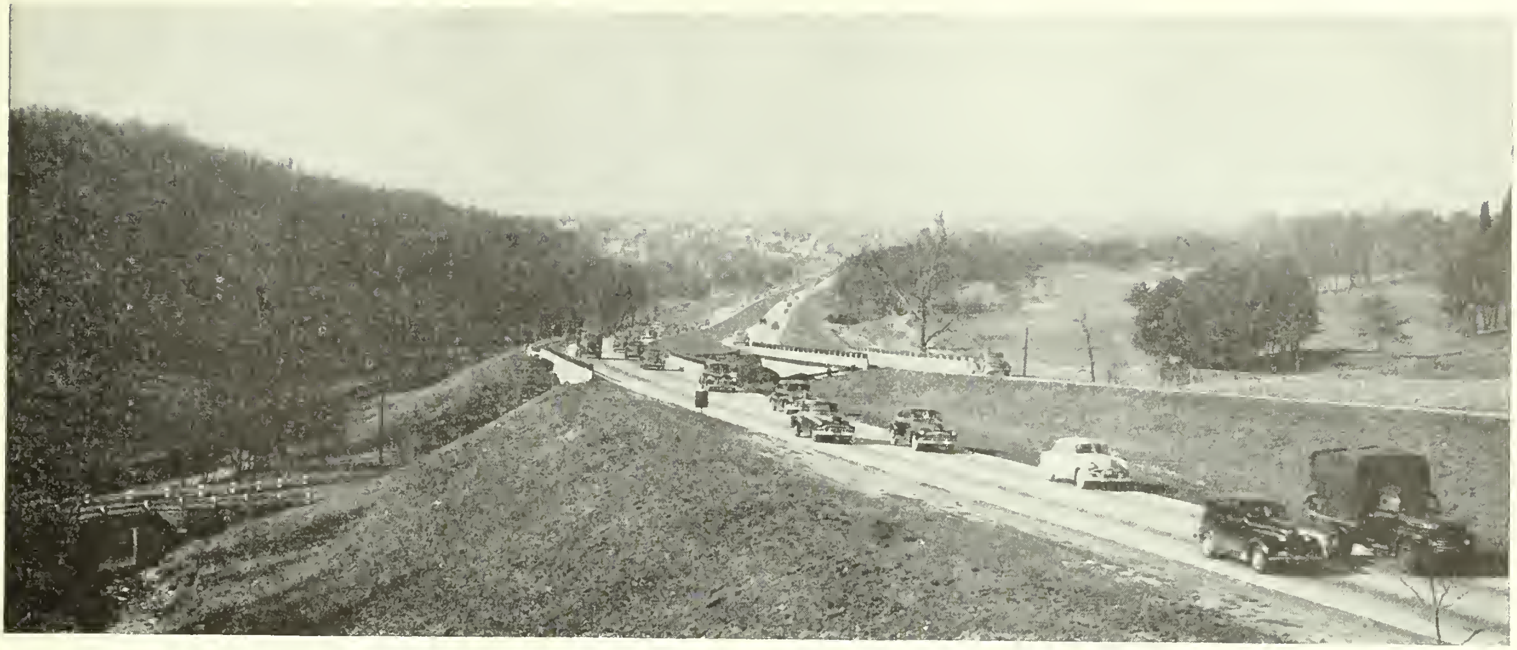

Figure 1.—End of the morning peak traffic on the Shirley Highway.

The Effect of Travel Time and Distance on Freeway Usage

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by DAREL L. TRUEBLOOD
Highway Transport Research Engineer

Until recently, few data have been available concerning the factors that influence motorists in choosing routes of travel in urban areas. Although a number of different factors may be involved, the effect of travel time and travel distance seem especially desirable for initial study because they are items that can be measured with reasonable accuracy on any route and their effect on the action of traffic related to the usage of that route. The relation of these two factors to the usage of the Shirley Highway, a freeway in Arlington and Fairfax Counties, Virginia, has been made a subject of study and the general findings are reported in this paper.

Before definite conclusions, acceptable for wide application, can be reached, the results from this study must be integrated with those from similar studies now underway in other urban areas. In general, though, it appears that motorists regard travel time as more important than distance in choosing a route of travel. Of all the trips on the Shirley Highway that were examined, only 38 percent saved distance while 81 percent saved time.

That motorists are also influenced to some extent by factors other than travel time and distance is evidenced by the fact that 19 percent of the Shirley Highway trips that were examined lost both time and distance by using that highway. Furthermore, of all the trips studied that could have saved both time and distance on the Shirley Highway, 10 percent used an alternate route instead. The reasons why these two groups of motorists chose the routes of travel they did is a question that can only be resolved by additional research.

THE NEED for increased capacity of our urban highway systems is recognized equally by the average citizen and the highway engineer, since both are familiar with the continued increases in vehicles and travel, the growing number of accidents, and the economic loss due to traffic congestion. To be really effective, modernization must be on a scale sufficiently generous to permit the safe, rapid flow of the large volumes of traffic that stream daily into and out of our metropolitan areas and move from point to point within these areas. This requires more than minor improvement of existing inadequate streets. In many instances, new controlled-access expressways to provide increased capacity will be needed.

Accepting this as a premise, the highway engineer charged with the responsibility of planning these new systems is immediately confronted with three questions: (1) What is the capacity of the existing street system? (2) How much additional capacity is needed to serve adequately the present and future over-all traffic demand? (3) What new facilities will be required and what volume of traffic may be expected on them?

Data in the *Highway Capacity Manual*[1] are available for determining an answer to the first question. The second question can be answered through the use of origin and destination traffic study techniques developed during the past 5 or 6 years, when used in conjunction with estimates of future urban growth. The highway engineer is not so fortunate when it comes to answering the third question, however, for he has not been able to estimate with confidence the amount of traffic a new facility will attract from existing streets. Data upon which to base an answer to this question have been lacking. The delay in undertaking research on this subject may be attributed, not to a failure to recognize the need of such information, but rather to a lack of urban expressways upon which data of an empirical nature could be collected.

With attention focused more directly on the improvement and construction of highway transportation facilities in urban areas during the past few years, more projects suitable for this type of research have become available for study. Interest has recently been stimulated through the efforts of the Subcommittee on Factual Surveys of the American Association of State Highway Officials and studies have now been undertaken in several different cities. Such a study was conducted during the summer of 1950 on the urban portion of the Shirley Highway, a freeway in Arlington and Fairfax Counties, Va. The Traffic and Planning Section of the Virginia State Department of Highways assisted in this study by making the field interviews.

Conclusions

Certain general conclusions are revealed from the data collected and analyzed in this study, but these findings must be integrated with those from similar studies now underway in other urban areas before definite conclusions acceptable for wide application can be reached. Considering all of the passenger-car trips between the origins and destinations which might result in freeway usage:

1. A general relation is found between the proportion of trips via the freeway and travel-distance ratios, but the variation in usage of the freeway is quite large when the distance by way of the freeway is approximately equal to or slightly greater than that by an alternate route.
2. Although there is some difference in the proportional use of the freeway for trips of different lengths, the difference does not appear to be greatly significant insofar as traffic assignment is concerned.
3. Good correlation is found between the proportion of trips via the freeway and the ratio of travel time via that route to the time via the most favorable alternate route.
4. A slightly better correlation than any other explored was found between the proportion of trips via the freeway and the actual time that is saved or lost in traveling by way of the freeway as compared with that by an alternate route.
5. Motorists, in traveling from one point to another in the study area, apparently regard travel time as more important than distance in selecting a route of travel. Of all the trips on the freeway that were examined, only 38 percent saved distance by using that route, while 81 percent saved time.

The Problem

The complexity of travel in urban areas is known to all who study city traffic and city planning. Parallel streets offer many alternate routes of travel and motorists in their daily travel do not hesitate to change routes in order to avoid one which has become congested or otherwise unattractive to use. It is common knowledge that they will go considerable distances out of their way in order to reach attractive, free-flowing arterials of modern design.

Origin and destination traffic studies provide information concerning the total number of vehicles passing from one zone to another in urban areas but this knowledge, within itself, is not sufficient. It is essential, for purposes of design and for other reasons, to estimate the number that will be attracted to a new arterial route when it is constructed. The making of such traffic-volume estimates is commonly referred to as traffic assignment. Since the major proportion of the traffic that will use a new route will usually consist of vehicles diverted from the existing street system, the extent to which they can be diverted to the new route and the factors which influence that diversion are of vital importance to those who have the responsibility for planning adequate highway facilities.

In the absence of factual data there is at present some disagreement among highway engineers regarding the reasons a motorist chooses one route instead of another. Consequently there is lack of agreement regarding the proper basis upon which to make traffic assignments. Travel time, travel distance, length of trip, ability to keep moving, safety, convenience, economy, habit, and other factors may enter into the choice. Very little is known, as yet, about the individual effect of any one of these factors. Some engineers consider travel time alone to be the most significant; others believe travel time and travel distance to be equally important; opinions concerning the significance of the other factors are usually indefinite and varied.

Although it is possible that a number of different factors may be involved, travel time and travel distance appear the most promising for initial study because they are measurable items. Both travel time and distance can be determined with reasonable accuracy on any route, even a new one proposed for construction. Furthermore, if a definite relation exists between either one or a combination of these two factors and the choice of routes, that relation, when established, will provide a practicable basis upon which traffic assignments can be made with confidence. It is, therefore, the effect of these two factors on the usage of the Shirley Highway that is explored in this study. The findings reported herein pertain strictly to diverted traffic and are limited to passenger-car travel.

The Henry G. Shirley Memorial Highway extends southwesterly through Arlington and Fairfax Counties in Virginia from a point near the Pentagon. At the north end it connects with a network of expressways serving that building, and, via this network, with three bridges crossing the Potomac River to Washington, D. C. Access to either the Shirley Highway or several alternate routes of travel from any one of the three bridges is readily available by way of this network.

Shirley Highway Selected for Study

The Shirley Highway is a four-lane, divided freeway with full control of access throughout its entire length. Each lane is 12 feet wide, and a 30-foot grass median separates the opposing directions of travel. The posted speed limit for passenger cars in Arlington County is 50 miles per hour, while in Fairfax County it is 55 miles per hour. Through trucks were prohibited from using the route at the time of this study.

The length of the freeway is approximately 18 miles from its beginning near the Pentagon to the point where it joins U S 1 south of Alexandria. Slightly more than 5 miles at the north end pass through a residential area suitable for a study of the type herein reported. Within the 5-mile section are five traffic interchanges where vehicles may enter or leave the freeway. At the time of this study, the average weekday traffic volume near the middle of the study section was about 30,000 vehicles per day, including both directions of travel.

The cover illustration shows the Shirley Highway, looking north from a point just north of the Glebe Road interchange. This picture, taken in September 1950 at 5:30 p.m., shows the heavy outbound movement of traffic during the evening peak period of travel. Figure 1 is a view in the opposite direction, looking south from the Arlington Ridge Road interchange. This picture was taken in April 1950 about 9 a.m., just after the inbound morning peak had passed. Some of the populous residential area served by the freeway is shown in the background.

There are three principal alternate routes of travel, in addition to the Shirley Highway, which serve the area selected for study. These are the Mount Vernon Me-

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[1] *Highway Capacity Manual*, by the Committee on Highway Capacity, Department of Traffic and Operations, Highway Research Board. Published by the Bureau of Public Roads.

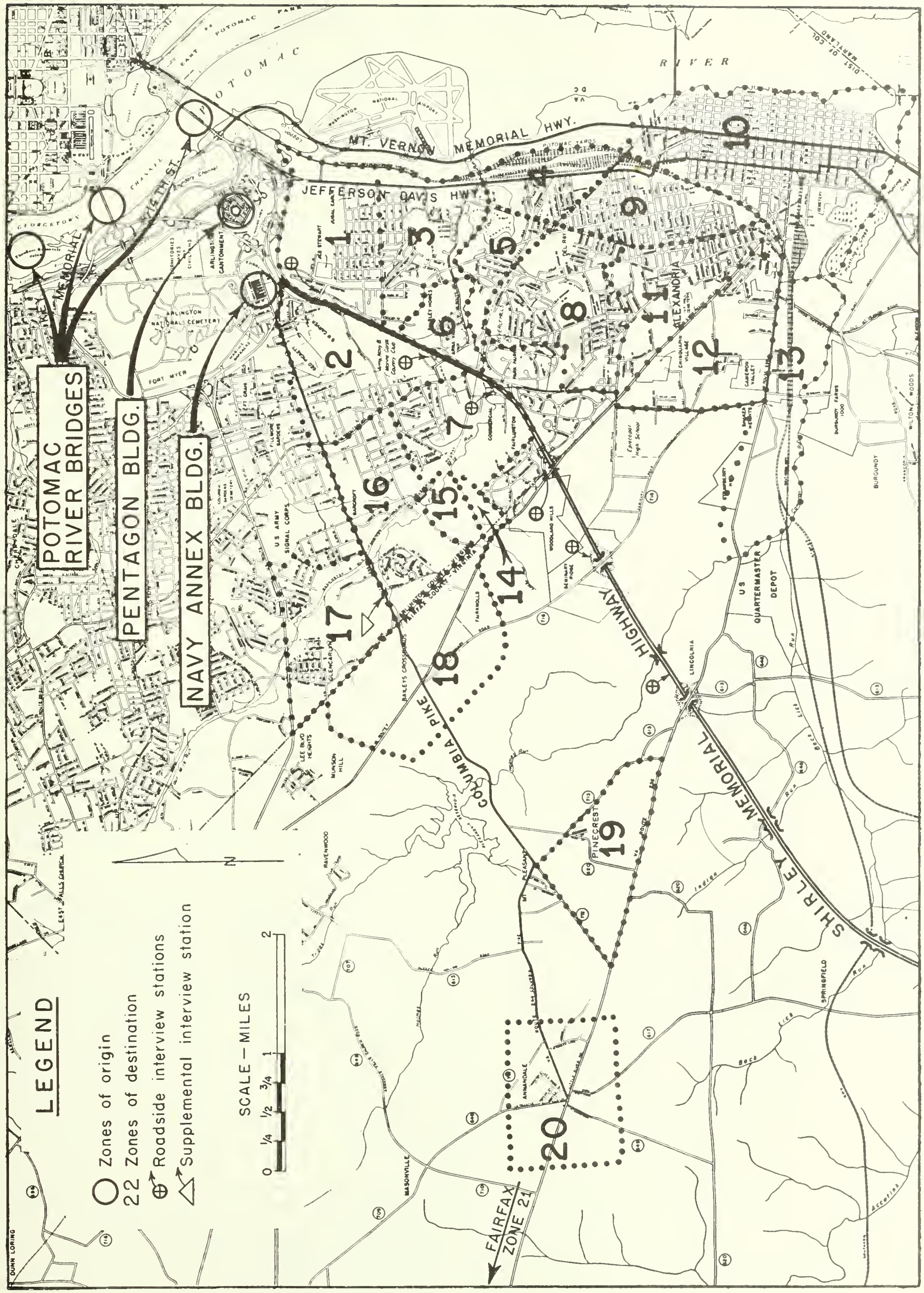

Figure 2.—Map of the study area.

Table 1.—*Origin, destination, travel time, and travel distance of trips studied*

Zone of destination	Number of trips			Travel time				Travel distance		
	Total	Via freeway								
		Number	Percentage of total	Via freeway	Via alternate	Ratio, freeway to alternate	Time differential	Via freeway	Via alternate	Ratio, freeway to alternate
ORIGIN AT FOURTEENTH STREET BRIDGE										
			Percent	*Min.*	*Min.*		*Min.*	*Miles*	*Miles*	
1	785	170	21.7	6.3	4.6	1.37	−1.7	2.8	1.9	1.47
2	890	52	5.8	10.3	7.0	1.47	−3.3	4.4	3.3	1.33
3	424	131	30.9	7.0	6.0	1.17	−1.0	3.4	2.4	1.42
4	332	19	5.7	11.0	7.0	1.57	−4.0	5.1	3.1	1.65
5	576	496	86.1	8.0	8.8	.91	.8	3.7	3.2	1.16
6	634	370	58.4	7.3	8.3	.88	1.0	4.0	3.7	1.08
7	1,192	1,172	98.3	6.5	13.5	.48	7.0	3.9	4.9	.80
8	860	478	55.6	9.5	10.0	.95	.5	4.9	4.2	1.17
9	675	148	21.9	12.2	9.9	1.23	−2.3	5.2	4.3	1.21
10	2,308	188	8.1	18.0	13.0	1.38	−5.0	7.7	5.2	1.48
11	467	193	41.3	12.0	13.0	.92	1.0	6.2	4.9	1.27
12	108	100	92.6	12.2	16.4	.74	4.2	6.2	5.9	1.05
13	176	43	24.4	14.0	21.0	.67	(1)	(1)	(1)	(1)
14	57	55	96.5	9.8	17.2	.57	7.4	5.4	6.9	.78
15	89	84	94.4	10.0	17.0	.59	7.0	5.5	6.8	.81
16	700	151	21.6	12.7	10.7	1.19	−2.0	4.9	4.3	1.14
17	177	18	10.2	14.7	11.7	1.26	−3.0	6.9	4.7	1.47
18	322	169	52.5	12.6	12.9	.98	.3	6.4	5.4	1.19
19	72	60	83.3	14.6	18.6	.78	4.0	8.4	8.2	1.02
20	291	196	67.4	17.7	20.2	.88	2.5	10.2	9.4	1.09
21	60	37	61.7	27.3	29.8	.92	2.5	16.3	15.5	1.05
ORIGIN AT MEMORIAL BRIDGE										
1	242	82	33.9	6.8	6.9	0.99	0.1	3.2	3.2	1.00
2	382	22	5.8	10.8	7.4	1.46	−3.4	4.8	3.5	1.37
3	184	84	45.7	7.5	8.3	.90	.8	3.7	3.7	1.00
4	200	15	7.5	11.5	9.2	1.25	−2.3	5.4	4.5	1.20
5	192	123	64.1	8.5	11.1	.77	2.6	4.0	4.6	.87
6	198	156	78.8	7.8	11.3	.69	3.5	4.3	4.6	.93
7	322	321	99.7	7.0	13.9	.50	6.9	4.3	5.3	.81
8	346	284	82.1	10.0	12.3	.81	2.3	5.2	5.5	.95
9	188	62	33.0	12.7	12.1	1.05	−.6	5.6	5.5	1.02
10	560	65	11.6	18.5	15.2	1.22	−3.3	8.1	6.9	1.17
11	65	44	67.7	12.5	15.3	.82	2.8	6.6	6.2	1.06
12	153	105	68.6	13.0	16.5	.79	3.5	6.6	7.2	.92
[2]13	27	36	...	...	...	...	...	...	...	...
14	28	28	100.0	10.3	17.6	.59	7.3	5.7	7.2	.79
15	27	27	100.0	10.5	17.4	.60	6.9	5.8	7.1	.82
16	406	60	14.8	13.2	11.1	1.19	−2.1	5.3	4.5	1.18
17	101	5	5.0	15.2	12.1	1.26	−3.1	7.1	4.9	1.45
18	377	169	44.8	13.1	13.3	.98	.2	7.2	5.9	1.22
19	43	30	69.8	15.0	15.9	.94	.9	9.2	7.3	1.26
20	281	199	70.8	18.1	20.6	.88	2.5	11.1	9.9	1.12
21	54	49	90.7	27.8	30.3	.92	2.5	17.1	16.0	1.07
ORIGIN AT KEY BRIDGE										
1	184	66	35.9	9.3	8.4	1.11	−0.9	4.2	3.9	1.08
2	198	8	4.0	13.3	9.9	1.34	−3.4	5.7	4.4	1.30
3	105	23	21.9	10.0	9.8	1.02	−.2	4.6	4.5	1.02
4	49	10	20.4	14.0	10.8	1.30	−3.2	6.3	5.2	1.21
[2]5	60	137	...	...	...	...	...	...	...	...
6	86	73	84.9	10.3	12.8	.80	2.5	5.2	5.7	.91
[2]7	141	157	...	...	...	...	...	...	...	...
[2]8	2	32	...	...	...	...	...	...	...	...
9	113	57	50.4	15.2	14.7	1.03	−.5	6.5	6.4	1.02
10	290	46	15.9	21.0	17.8	1.18	−3.2	8.9	7.7	1.16
11	65	30	46.2	15.0	16.8	.89	1.8	7.5	7.0	1.07
12	28	18	64.3	15.5	19.0	.82	3.5	7.5	7.7	.97
[2]13	0	11	...	...	...	...	...	...	...	...
14	20	20	100.0	12.8	18.7	.68	5.9	6.7	7.3	.92
15	24	22	91.7	13.0	18.5	.70	5.5	6.8	7.2	.94
16	115	17	14.8	15.7	12.2	1.29	−3.5	6.1	4.6	1.33
17	119	0	0	17.7	11.0	1.61	−6.7	8.0	5.8	1.38
18	23	11	47.8	15.6	15.8	.99	.2	8.0	6.7	1.19
19	7	4	57.1	17.6	19.2	.92	1.6	10.0	7.3	1.37
[3]20	...	...	...	...	...	...	...	...	...	...
[3]21	...	...	...	...	...	...	...	...	...	...
ORIGIN AT PENTAGON										
			Percent	*Min.*	*Min.*		*Min.*	*Miles*	*Miles*	
1	140	55	39.3	4.0	3.8	1.05	−0.2	1.8	1.5	1.20
2	141	19	13.5	7.8	6.2	1.26	−1.6	3.4	2.4	1.42
3	64	56	87.5	4.8	5.8	.83	1.0	2.3	2.1	1.10
4	29	6	20.7	8.5	7.4	1.15	−1.1	4.1	2.8	1.46
5	234	57	24.4	6.3	7.3	.86	(1)	(1)	(1)	(1)
[2]6	58	98	...	...	...	...	...	...	...	...
[2]7	220	424	...	...	...	...	...	...	...	...
8	398	241	60.6	7.9	9.2	.86	1.3	3.9	3.8	1.03
9	75	40	53.3	10.5	9.8	1.07	−.7	4.3	4.0	1.08
10	232	66	28.4	15.1	13.6	1.11	−1.5	6.8	4.8	1.42
[3]11	0	59	...	...	...	...	...	...	...	...
12	65	46	70.8	11.1	14.3	.78	3.2	5.3	5.5	.96
13	30	21	70.0	12.3	17.0	.72	4.7	5.4	6.9	.78
14	59	59	100.0	7.8	16.4	.48	8.6	4.5	6.0	.75
15	50	50	100.0	8.0	16.2	.49	8.2	4.6	5.9	.78
16	148	69	46.6	9.3	10.0	.93	.7	4.0	3.2	1.25
17	154	15	9.7	14.0	11.3	1.24	−2.7	5.7	3.7	1.54
18	284	169	59.5	10.9	12.1	.90	1.2	5.7	4.5	1.27
19	23	19	82.6	12.9	17.7	.73	4.8	7.6	7.2	1.06
20	139	113	81.3	16.0	19.3	.83	3.3	9.3	8.5	1.09
21	17	15	88.2	24.9	29.5	.84	4.6	15.4	14.6	1.05
ORIGIN AT NAVY ANNEX BUILDING										
1	65	54	83.1	3.7	4.9	0.76	1.2	1.7	2.1	0.81
2	123	8	6.5	6.9	4.5	1.53	−2.4	2.9	1.5	1.93
[2]3	17	28	...	...	...	...	...	...	...	...
4	24	8	33.3	8.3	8.0	1.04	−.3	3.7	3.6	1.03
5	67	41	61.2	6.1	8.3	.73	2.2	2.5	3.8	.66
6	74	65	87.8	5.3	8.0	.66	2.7	2.6	3.2	.81
7	143	140	97.9	5.0	11.0	.45	6.0	2.6	3.1	.84
8	67	53	79.1	7.6	10.7	.71	3.1	3.5	4.0	.88
9	49	39	79.6	10.2	11.7	.87	1.5	3.8	4.7	.81
[2]10	0	50	...	...	...	...	...	...	...	...
[2]11	0	15	...	...	...	...	...	...	...	...
[2]12	0	5	...	...	...	...	...	...	...	...
[2]13	0	6	...	...	...	...	...	...	...	...
14	7	6	85.7	8.0	13.0	.62	5.0	4.2	4.9	.86
15	11	11	100.0	8.2	12.8	.64	4.6	4.3	4.8	.90
16	112	13	11.6	8.9	7.5	1.19	−1.4	3.6	2.3	1.57
17	65	0	0	13.5	8.3	1.63	−5.2	5.2	2.7	1.93
18	72	14	19.4	10.5	9.6	1.09	−.9	5.4	3.5	1.54
19	2	2	100.0	12.5	15.2	.82	2.7	7.2	6.4	1.13
20	28	15	53.6	15.6	16.8	.93	1.2	8.9	7.7	1.16
21	5	2	40.0	25.3	26.4	.96	1.1	15.0	13.8	1.09

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[1] Not included for analysis because percentage of traffic using freeway, when related to travel time ratio, falls far out of general range of other data.

[2] Not used in analysis because of inadequate samples and uncertainties in adjustment of 1948-50 travel.

[3] Insufficient data available to make an estimate of the total zone-to-zone movement.

morial Highway, Jefferson Davis Highway (U S 1), and Columbia Pike. The latter two are typical city-street arterials with the usual signalized intersections, commercial development, and accompanying traffic congestion. The Mount Vernon Memorial Highway, being in the nature of a parkway, is more attractive to travel than the other two. There are, of course, many city streets of lesser importance than the three arterials named that also serve the area.

The map in figure 2 shows the general area of the study and the location of the Shirley Highway in relation to the alternate routes and the city streets serving the area.

Study Procedure

The procedure adopted utilizes the origin and destination data collected in the Washington metropolitan area transportation survey, combined with those obtained from roadside interviews made at points of exit along the Shirley Highway. With these data at hand, supplemented with travel time and distance measurements, it was possible to relate the percentage of traffic using the freeway between certain origins and destinations with the ratio of travel time or distance by way of the freeway to that by an alternate route.

The Washington transportation survey provided information concerning the total number of passenger cars moving from one zone to another regardless of the route traveled. This survey was conducted during the summer and fall of 1948 by the home-interview method, a 5-percent sample of the dwelling units being interviewed.

In order to adjust for the larger volume of traffic in 1950, the zone-to-zone movements from the 1948 survey were uniformly increased by 20 percent. The amount of this increase was estimated from July and August traffic counts made in 1948 and in 1950 at 10 automatic recorder stations in the metropolitan area and, also, from a comparison of the travel in 1948 with that in 1950 between the city of Washington and the Fairlington apartment development. Fairlington is a large residential development, containing about 3,600 dwelling units and housing approximately 12,000 people, located directly on the Shirley Highway at the Arlington-Fairfax county line. Practically all of the dwelling units were occupied in 1948 and also in 1950, so a direct comparison of the traffic data was possible.

An increase of 15.2 percent was found at the recorder stations and an increase of 23.1 percent in the Washington-to-Fairlington traffic. It was decided to give slightly more weight to the latter, and a 20-percent increase was selected as reasonable for the uniform expansion. In addition to this expansion, certain zone-to-zone movements were increased by appropriate supplemental factors to account for unusual changes in population, employment, and commercial development known to have occurred since 1948.

The number of passenger cars using the Shirley Highway in going from one zone to another was determined from data collected at roadside interview stations. Interview stations were established on all exit ramps along the freeway from its beginning near the Pentagon to the end of the study area near the Lincolnia interchange (Virginia Route No. 236). This required five interview stations. At the end of the study area, just north of the Lincolnia interchange, a station was established directly on the Shirley Highway and a sample of all outbound passenger cars passing this point was interviewed. Also, to assist in determining the total travel to some of the outlying zones, a supplemental interview station was established on Columbia Pike. The location of these stations is indicated by distinctive symbols on the map in figure 2.

Each station was operated for a period of 16 hours on a weekday, 6 a.m. until 10 p.m., by an experienced crew of the Traffic and Planning Section of the Virginia Department of Highways. During the time of this study, July 19 to August 3, 1950, an average of 23,249 passenger cars passed the six interview stations along the Shirley Highway in the 16-hour period. Interviews were obtained from the drivers of 15,667 of these vehicles, or about 67 percent.

The data were coded, punched on tabulating cards, and appropriate factors applied by hourly periods to expand the information to an average 24-hour weekday representative of the period of the study. A tabulation was then prepared showing the zone of origin and the zone of destination of all outbound passenger-car drivers using the freeway.

In order to investigate the effect of travel time on the choice of route, it was necessary to determine the time required to travel between points of origin and destination via the freeway and via the alternate routes. A comprehensive travel-time map prepared for the Washington transportation survey provided much useful information in this connection. Check runs by the "floating-car" method were made on the freeway and on the principal alternate routes to test for differences between 1948 and 1950 travel time. The times recorded represent average peak-hour conditions on a weekday and were measured to the center of population of each zone.

As with the travel-time measurements, the distances were measured to the center of population of each zone via the freeway and via the shortest alternate route. In each case the mileage was scaled from a 1:24,000-scale map of the study area. A number of field checks made with a passenger car showed close agreement between the scaled distances and the odometer readings.

The time and distance measurements as well as the traffic volumes between points of origin and destination used in the study are shown in table 1.

Method of Analysis

Since a part of the basic data for this study was derived from a 5-percent sample of travel, it follows that zone-to-zone movements of very low volume are not suitable for use. For this reason, it was decided to consider the city of Washington and its Maryland suburbs as a single zone for purposes of this study. All trips originating therein and destined to zones in the study area must cross one of the three Potomac River bridges designated in figure 2. Thus, for purposes of this analysis, these bridges have been considered as points of origin for all trips beginning on the Washington side of the Potomac River. While information relative to the actual bridge crossed was not available, groups of trips were assigned to the most logical crossing according to their zone of origin and zone of destination.

The Pentagon and the Navy Annex Building are major traffic generators on the Virginia side of the Potomac River and these, in addition to the three bridges—Fourteenth Street, Memorial, and Key—spanning the Potomac River, comprise the five points of origin used in the study.

By reviewing the tabulation of passenger cars that used the freeway it was possible to determine the zones in Arlington and Fairfax Counties that were destinations of a substantial number of vehicles using that facility. Twenty-one such zones were tentatively selected. The findings reported in this article are based on an analysis of the travel from the 5 points of origin to these 21 zones of destination. In total, 105 different groups of trips were examined, but 15 were found to be unsatisfactory for use because of inadequate samples, uncertainties in adjustment of 1948 travel to 1950, or for some other reason, and these movements were disregarded in the analysis. Also disregarded in the analysis were trips originating outside of the Washington metropolitan area, since it was assumed that a majority of these trips would tend to follow marked routes regardless of the attractiveness of such routes for travel. In table 1 it will be noted that a few zone-to-zone movements of low volume were used, this being made possible through the use of the data collected at the supplemental

Table 2.—*Total number of trips studied*

	Number of trips	Percentage of total
		Percent
On freeway	8,152	39.0
On alternate routes	11,604	55.5
Subtotal	19,756	94.5
Not used	1,158	5.5
Total	20,914	100.0

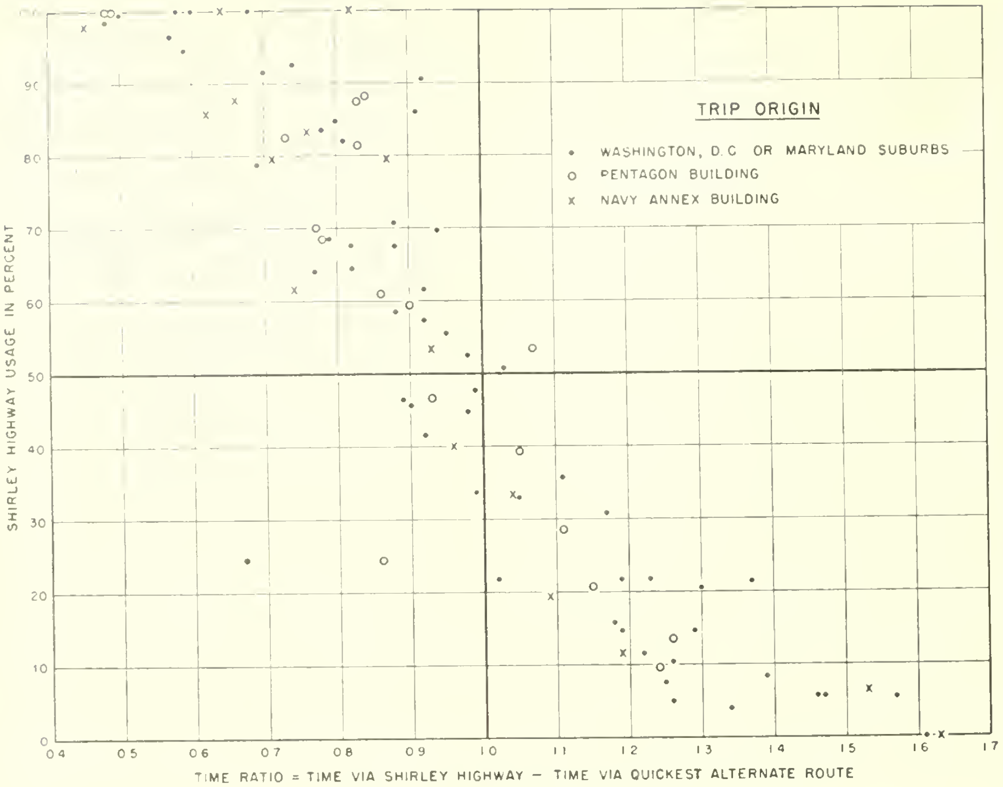


Figure 3.—Freeway usage in relation to time ratio.

roadside interview station on Columbia Pike.

Table 2 summarizes the total number of trips included for study and classifies them according to travel on the freeway, on alternate routes, and those that were not used.

Freeway Use Relation to Travel Time

Figure 3 shows the percentage of passenger-car traffic using the freeway for various travel-time ratios. The travel-time ratio in each case was derived by dividing the amount of time required to make the trip via the freeway by that required via the most favorable alternate route. Each symbol represents the group of trips beginning at one of the 5 points of origin and ending in one of the 21 zones of destination. For example, the small circle near the middle of the chart in the upper right quadrant (1.07 time ratio and 53 percent freeway usage) represents the group of trips beginning at the Pentagon and ending in zone 9. Table 1 shows the total number of trips in this movement to be 75, of which 40 used the Shirley Highway. The dot to the left and slightly below the circle, but also in the upper right quadrant, represents a movement of 113 trips beginning on the Washington side of the Potomac River, crossing Key Bridge and, as it happens, also ending in zone 9. Fifty-seven of these trips used the Shirley Highway.

In total, the 56 dots on the chart represent 16,970 trips originating on the Washington side of the Potomac River, the 18 small circles represent 2,282 trips originating at the Pentagon, and the 16 crosses represent 914 trips originating at the Navy Annex Building. Included are two groups totaling 410 trips that were not used in subsequent analyses because they fall so far out of the general range of the other points. The symbols for these groups are in the 20–30 percent usage of the chart, to the left of 0.9 time ratio.

Although, as expected, there is some scatter in the points, they seem to fall within a reasonably close band all the way across the chart. The general pattern suggests the probability of a relation that may be expressed in terms of an S curve. No attempt was made to fit a curve to the points on this chart, however, because they represent different values insofar as the number of trips is concerned.

To arrive at a weighted mean, and also to reduce the number of points, the data were summarized by combining those movements which have the same travel-time ratio within increments of one-tenth (for example, 0.96 to 1.05) and computing the percentage of the total trips of these combined movements that used the freeway. The results of this summarization are shown by small circles in figure 4. The position of these circles clearly indicates a definite relation between travel-time ratios and freeway usage. While all of the circles do not fall directly on a smooth S curve, especially at each extremity, those near the center fit remarkably well. This may be due, in part, to the greater number of trips represented by those points. The position of the five circles near the center (0.8 to 1.2 time ratios) was determined from a study of 11,205 trips, while the position of the remaining seven circles was determined from 8,551 trips. The curve in figure 4 (and all others shown in this article) was fitted by inspection.

From this curve it is apparent that

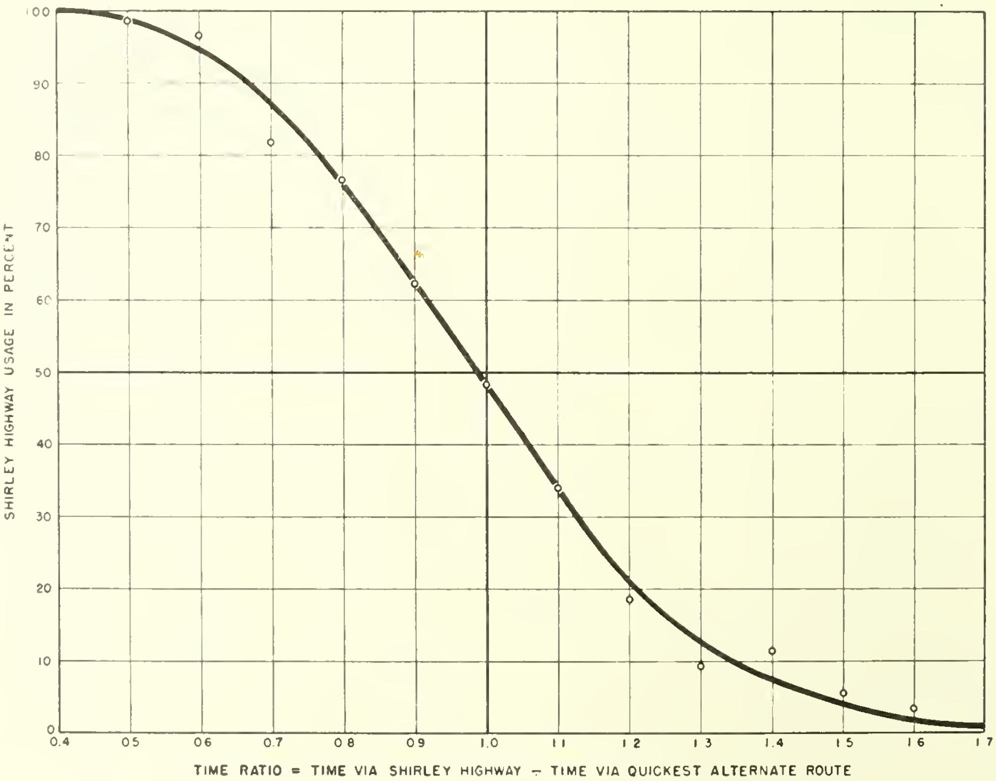


Figure 4.—Curve for freeway usage in relation to time ratio.

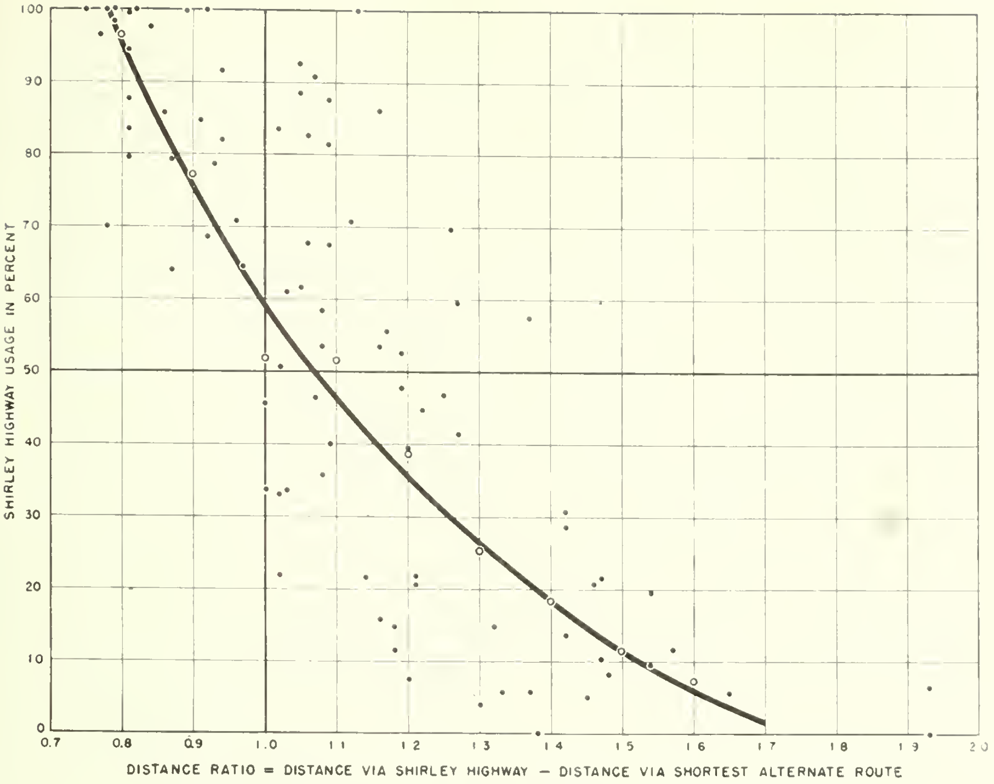


Figure 5.—Freeway usage in relation to distance ratio.

practically all of the motorists use the freeway when the travel time by way of that route is less than 0.4 of that by way of the most favorable alternate route. At the other extreme, when travel time via the freeway is greater than 1.7 times that via an alternate, almost all of the motorists use the alternate route. When the travel time is the same on the freeway as that on an alternate route, approximately 48 percent of the drivers choose the freeway even though it is necessary to travel additional distance in order to do so.

Freeway Use Relation to Distance

Figure 5 shows the percentage of passenger-car traffic using the freeway for various travel-distance ratios. The same general procedure was used in developing this chart as was used for the one shown in figure 4. In this case, however, the scatter of the points is much greater, especially near the middle of the chart between 1.0 and 1.4 distance ratios. Even though weighted means for groups of points with so much variation have little significance, the data were summarized by one-tenth distance ratios (shown by the small circles), and a curve fitted to these circles. Note that the shape of this curve, unlike that of the time-ratio S curve, is concave throughout.

It is evident from the data represented on this chart that practically all of the motorists use the freeway when the distance ratio is less than 0.8 and very few use it when the ratio is greater than 1.7. The usage when the distance ratios are between 1.0 and 1.1 varies from 22 to 92 percent. The exact reason for such a wide variation is unknown, although, from a supplementary analysis, it appears to be directly related to the quality of the traffic service provided by the alternate routes. The 22 movements comprising these trips were separated into two groups—one group having a choice of the freeway or an alternate providing reasonably good traffic service, and another group that must choose between the freeway or a relatively poor alternate. Of the first group, only 37.1 percent chose the freeway, while 66.6 percent of the second group chose that route. Furthermore, all except two of the eight movements included with the first group could travel via alternate routes in the same or less time than via the freeway, while all except one of the fourteen movements included with the second group could save time by using the freeway. Thus it is apparent that motorists making trips that are approximately equal in distance by the freeway and by an alternate route choose the former in greater proportions when travel time can be saved by doing so.

Freeway Use Relation to Time and Distance Combined

Since both the travel-time ratio and the distance ratio appear to bear some relation to the use of the freeway, it was decided to investigate a combination of the two. With this in mind, the distance ratio was divided by the time ratio for each group of trips, in effect giving a speed ratio, and the result plotted according to the percentage of passenger-car traffic using the freeway in each case. No correlation was found with this procedure. A second attempt was made to combine the two ratios, in which the time ratio and the distance ratio for each group of trips were multiplied and the product plotted according to the percentage of passenger-car traffic using the freeway in each case. Figure 6 shows the results of this combination after the detailed data were summarized by increments of one-tenth.

The tendency is more toward a straight line than the S curve found in connection with the time ratio (fig. 4). This is to be expected because, as a matter of mathematics, the product of the time and distance ratios tends to drop the relative position of the product curve below that of the time-ratio curve for each group of trips having a time ratio and a distance ratio both less than 1.0. Conversely, the tendency is to raise the relative position where either or both ratios are greater than 1.0.

While a relation between the freeway usage and the travel time-distance ratio product seems to exist, the correlation is not as good as that found with the time ratio alone. The relation shown in figure 6 is of general interest, but it appears to be less practicable and would provide less accurate results than the time-ratio curve if used as a basis for making traffic assignments.

Freeway Use Relation to Time Differential

Figure 7 shows the percentage of passenger-car traffic using the freeway based on the actual number of minutes motorists saved or lost by using that route as compared with an alternate. Here, as in the case of the travel-time ratio, the points fall within a reasonably close band which unmistakably suggests an S-curve relation.

The curve shown was drawn to fit the weighted means computed for each minute saved and each minute lost. As on previous charts, the weighted means are indicated by small circles. The resulting curve shows that where any group of motorists can save 8 or more minutes by using the freeway, they all choose that route. At the other extreme, a few motorists use the freeway even though they lose 4 or 5 minutes by doing so. When travel time via the freeway is the same as that via an alternate route, the curve shows that approximately 48 percent of the motorists choose the freeway. This agrees properly with the percentage use shown by the time-ratio curve when the travel times are equal.

An interesting feature of this relation is its tendency to group zone-to-zone movements according to length. The longer trips tend to fall near the extremities of the curve while the shorter trips are grouped nearer the middle. This is readily understandable, because it would be impossible to save or lose several minutes by using the freeway instead of an alternate

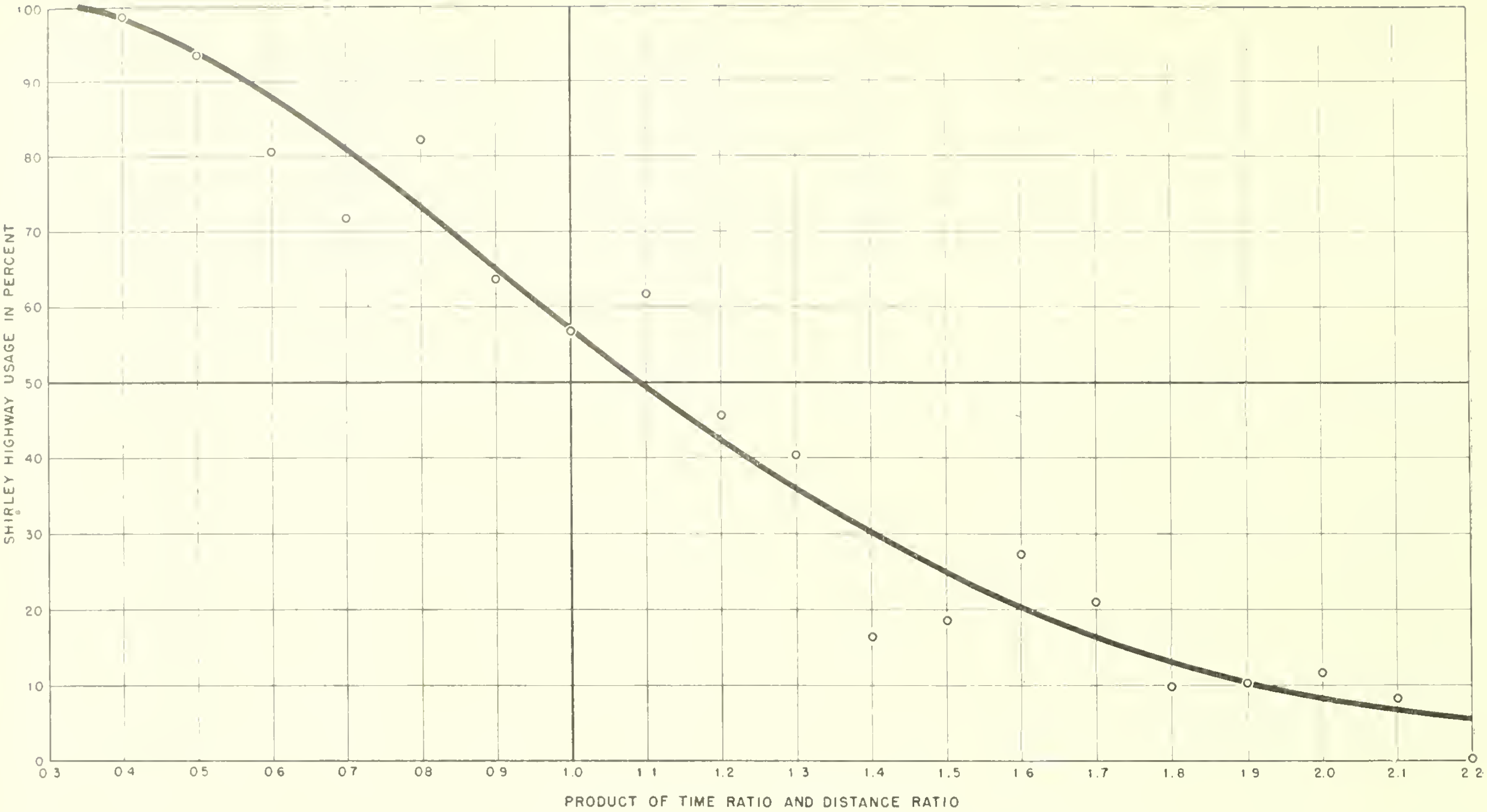


Figure 6.—Freeway usage in relation to product of time and distance ratios.

route in making short trips of only 5 or 10 minutes total duration. On the other hand, in making trips of 20 or 30 minutes duration, a time differential of several minutes would not be at all unlikely.

It is this tendency of trips to fall into groups according to length that results in somewhat better correlation between freeway usage and time differential than between freeway usage and time ratio. The reason for this difference is brought out in figure 8.

Freeway Use in Relation to Trip Length

Figure 8 shows the percentage of passenger-car traffic using the freeway, based on travel-time ratios, for three increments

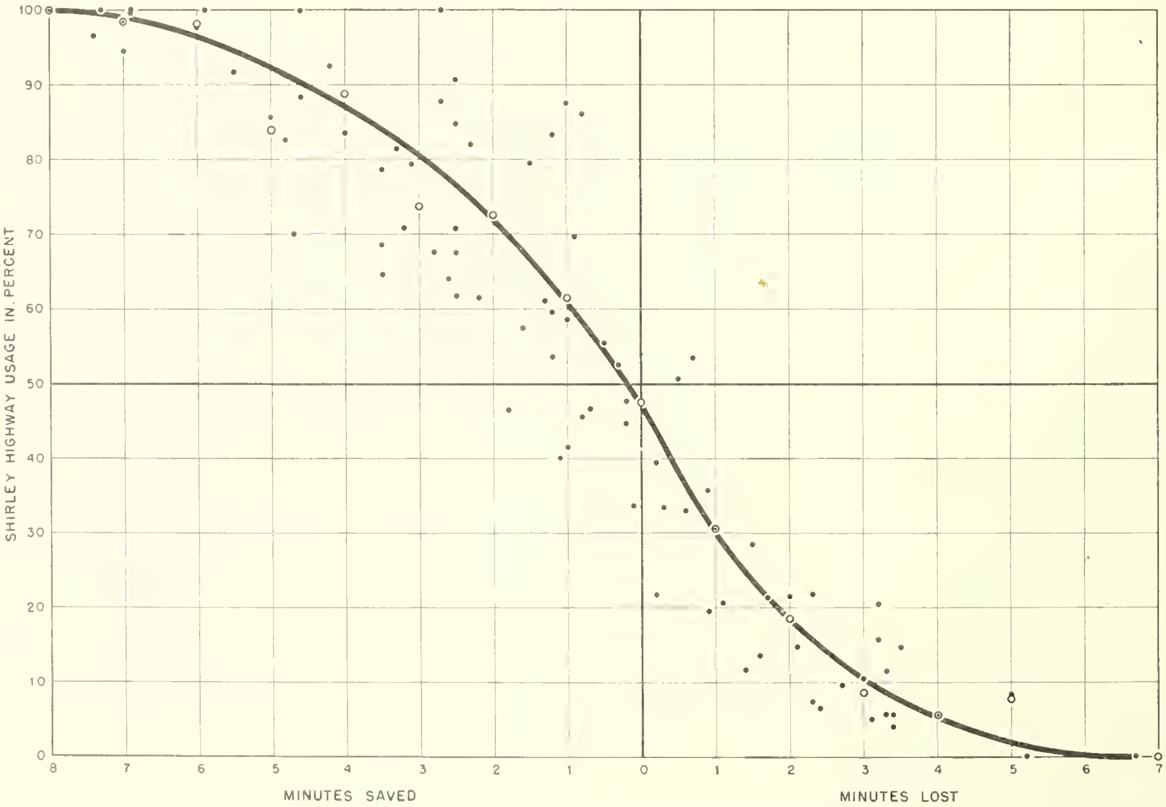


Figure 7.—Freeway usage in relation to time differential.

of travel distance: 4.0 miles and less, 4.1 to 6.4 miles, and 6.5 miles and greater. The distance by way of the freeway was used in grouping the trips into the three increments of length. The length in each case is the over-all distance between one of the five points of origin and one of the zones of destination. On this basis, the shortest trip included is 1.7 miles while the longest is 17.1 miles.

It is evident from the position of the three curves in figure 8 that, when the time ratio is less than 1.07, a greater percentage of the longer trips than of the shorter trips are on the freeway. When the time ratio is greater than 1.07, however, the position of the curves is reversed and a larger percentage of the shorter trips are on the freeway. For example, when the travel-time ratio is 0.7, these curves show that 89 percent of the longer trips are on the freeway and only 82 percent of the shorter ones. When the time ratio is 1.4, only 3 percent of the longer trips are on the freeway but there are 15 percent of the shorter ones.

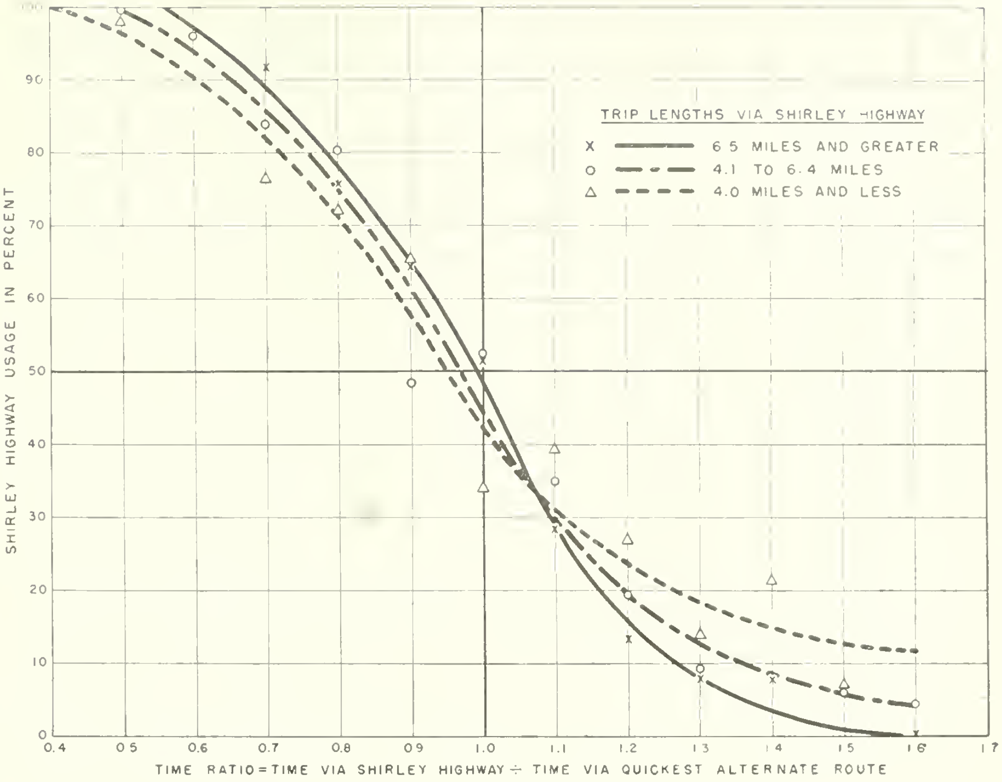


Figure 8.—Effect of trip length on freeway usage.

The explanation for this relation appears to be directly connected with the actual amount of time motorists can save, or will lose, in making trips of various lengths by one route as compared with that of another. This point can best be explained by an example. Assume a long trip to require 20 minutes by way of the freeway and a short one 5 minutes. If the time ratio is 0.7, motorists making the longer trip save 8.6 minutes by using the freeway while those making the shorter trip save only 2.1 minutes. The actual amount of time saved in the case of the longer trip is four times as great as that for the shorter trip. When the time ratio is 1.4, however, motorists lose 5.7 minutes in making the longer trip by way of the freeway, but only 1.4 minutes for the shorter one. In this case the loss in time is about four times as great for the longer trip.

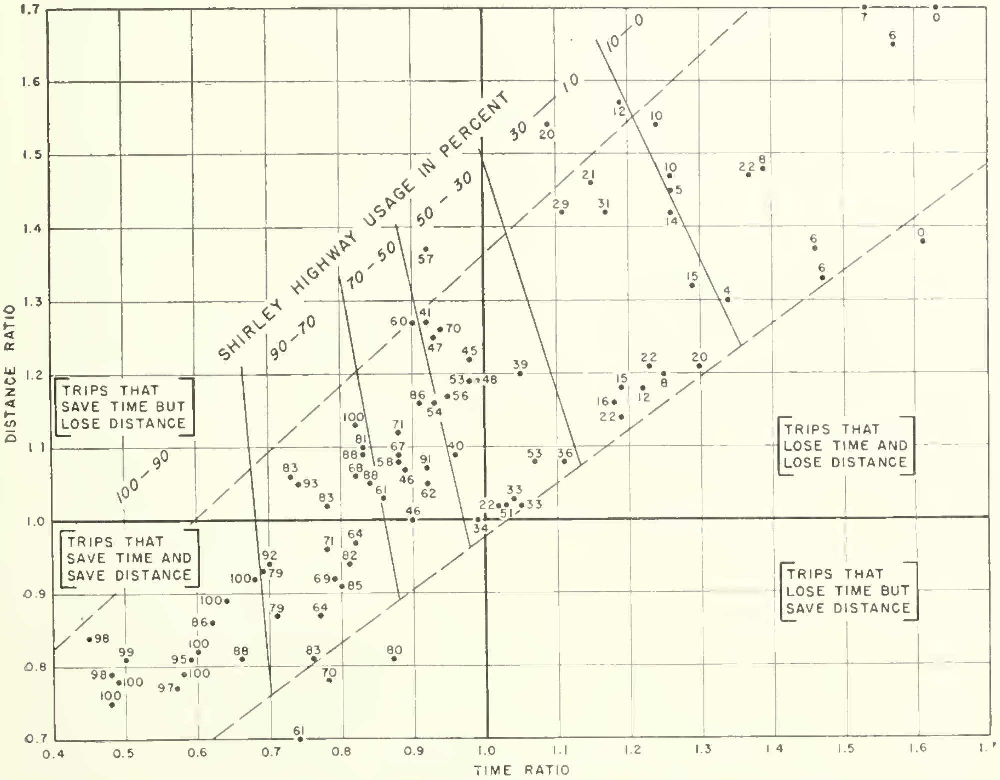


Figure 9.—Freeway usage in relation to time and distance ratios.

Thus it seems that motorists attach significance to the actual amount of time saved or lost in traveling from one point to another in urban areas (especially when the amount is substantial) as well as to the relative travel time by way of one route compared with that of another. It is quite possible, in the case of the shorter trips, that the increment of time saved or lost is so small that it is not only insignificant but probably unknown to motorists. This might further explain the reason for the relative position of the curves in figure 8.

If the travel-time ratio were the only criterion, the point at which the curves in figure 8 cross each other would occur at a ratio of 1.0 instead of 1.07. The positions of the curves show that, when the travel-time ratio is 1.0, the freeway is slightly more attractive to motorists making long trips than it is to those making short trips. The difference is so small in this case, however, that it could not be considered significant insofar as traffic assignment is concerned.

Freeway Use in Relation to Time and Distance Ratios

The percentage use of the freeway in relation to travel-time ratios and to travel-distance ratios has been shown on charts, separately, in figures 4 and 5. In figure 9 these two ratios and the percentage use of the freeway are shown on the same chart in order that the general relation of the three variables can be visualized and explored. Each dot on the chart represents a zone-to-zone movement and the adjacent numeral indicates the percentage of that movement using the freeway. These are plotted according to the time and distance ratios for each such movement.

The four statements shown in brackets on the chart, relative to saving or losing time and distance, apply to the four quadrants formed by the heavy vertical line at time ratio 1.0 and the heavy horizontal line at distance ratio 1.0. These statements refer to trips made by way of the freeway. Note that the lower right quadrant does not contain any dots. This is proper because, in this study, the average speed of travel on the freeway exceeds that on any alternate route; consequently, any zone-to-zone movement that would have lost time on the freeway would also have lost distance.

It is of interest that, in total, the freeway was used by 17 percent of the zone-to-zone movements plotted in the upper right quadrant, by 60 percent of those plotted in the upper left quadrant, and by 90 percent of those plotted in the lower left quadrant. Interpreting these percentages further, of the motorists whose trips were studied that would have lost both time and distance by using the freeway, 17 percent chose to do so, as did 60 percent of those who would have saved time but lost distance. On the other hand, of the motorists that could have saved both time and distance by using the freeway, 10 percent did not do so. This, again, seems to indicate the presence of factors other than time and distance that influence motorists in their choice of route.

The two dashed lines extending from the lower left to the upper right of the chart indicate the general range of time and distance ratios within which usage of the freeway occurs. The five solid lines sloping upward slightly to the left subdivide the area between the dashed lines into six segments. Each segment represents roughly a certain percentage range for use of the freeway as designated by the line of numerals extending diagonally across the chart above the upper dashed line, most of the percentages within a segment falling within the range indicated. It will be noted that the percentage of use gradually decreases from 100 percent at the lower left corner to zero at the upper right corner.

While it would have been desirable to have had more points from which to determine the slope of these five "contour" lines, the general direction of the third and fifth line from the left can be determined with reasonable accuracy from the points shown. To determine the slope of the three remaining lines, the third and fifth were extended to an intersection at a point above the chart and the remaining three lines projected back from that point of intersection as radii of a circle. This method seemed to conform with the data as nearly as any other logical one.

The slope of the resulting lines permits some interesting conjectures to be made. If all had turned out to be vertical this would have indicated that distance ratio has no effect at all on a motorist in his choice of route insofar as the factors of time and distance ratio are concerned. Conversely, had the lines assumed a horizontal position, it would indicate that time ratio has no effect. The lines as drawn suggest that both ratios affect the choice of route to some extent but, since the lines are more nearly vertical than horizontal, it follows that the time ratio is probably more significant than the distance ratio in this respect. Furthermore, since the slope of each line becomes greater as the percentage use of the freeway decreases, it suggests an increasing effect of the distance ratio as the time and distance ratios increase.

Statistical Comparison of Curves

As stated earlier, the principal purpose of this study is to show how travel time and travel distance affect the use of a freeway. The curves developed show the effects of these factors, but the correlation is not perfect in any of these cases. The points in some instances depart widely from the average relation expressed by the trend lines or curves fitted to the data. It is desirable to know the relative significance of the averages expressed by each curve before they can be used intelligently.

The standard error of estimate, or, more briefly, the standard error, offers a mathematical means of making this determination. The standard error serves not only as a general index of the significance of these curves, but also as a measure of the degree of accuracy of estimates based upon them. In other words, it measures the expected variability of estimated values from the actual values.

Therefore, in order to compare the curves developed in connection with time and distance ratios, and appraise their reliability for use in traffic assignment work, the standard error was computed for each curve. The results of these computations, which is the percentage variation that would not be exceeded more often than about one-third of the time, are summarized in table 3.

Of the four curves, the one based on time differential has the least standard error, while the one based on distance ratio has the greatest. It will be noted that the curve based on time ratio has a standard error only slightly greater than that of the time differential curve. This clearly indicates that the curves based on time differential and time ratio are approximately of equal reliability and that time differential and time ratio show the best correlation with the percentage use of the freeway. Either of these curves, if used for purposes of assigning zone-to-zone movements of traffic to the freeway, would provide results within 8 or 9 percent of the true values in at least two-thirds of the cases. This is satisfactorily within the accuracy of the basic data collected in origin-and-destination traffic studies conducted on the usual sampling basis. Moreover, the necessity of projecting traffic estimates into the future, with the attendant uncertainties, can readily introduce differences of greater magnitude than those that would result from the assignment of traffic on the basis of the time differential or travel-time ratio curves.

Table 3.—*Standard error of estimate*

Description of curve	Figure No.	Standard error
		Percent
Time ratio	4	8.66
Distance ratio	5	17.54
Product of time and distance ratios	6	11.14
Time differential	7	8.50

Highway Subdrainage

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

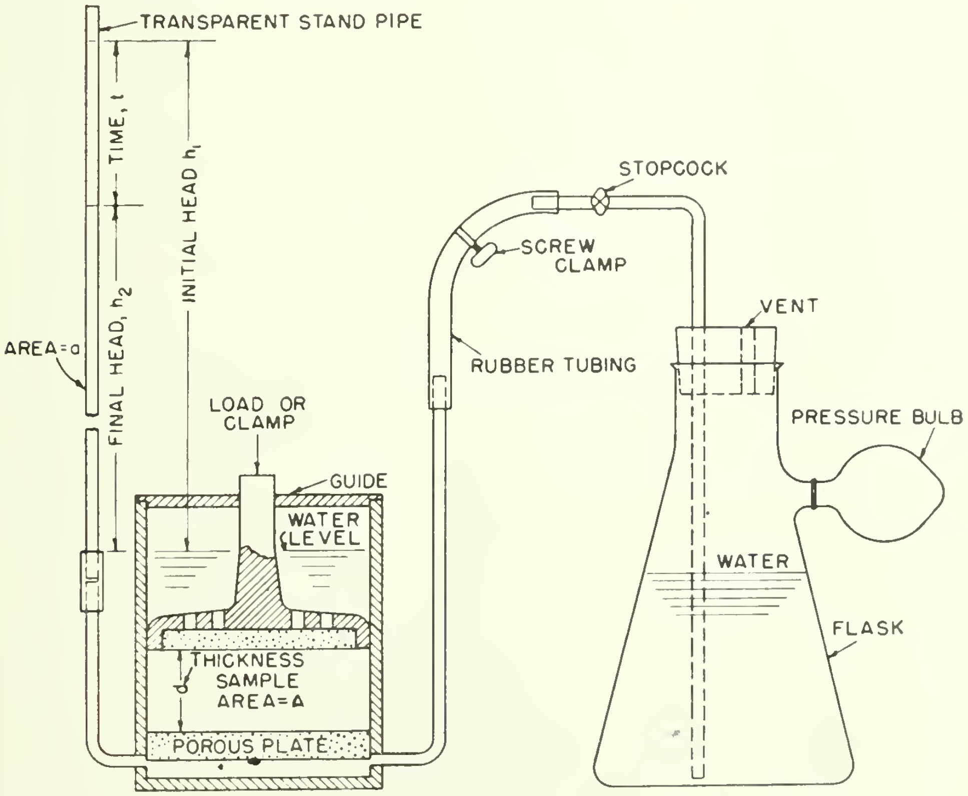
Reported by E. S. BARBER and C. L. SAWYER,
Highway Engineers

While the importance of subdrainage has long been recognized, highway drainage designs have generally been based on empirical rules. Although this report does not attempt to establish design criteria, it presents test methods and data on permeability and drainability of soil and indicates their application to highway subdrainage.

Results of permeability tests on various materials with appropriate apparatus show the importance of detailed specifications of procedure and materials and the extremely wide range of possible values. For instance, the permeability of a material depends on the method of its compaction as well as its density, gradation, and plasticity.

Substitution of test results in appropriate formulas illustrates the effect of boundary conditions and the coefficient of permeability on the rate of drainage. It is shown that the small gradients available for lateral drainage of base courses prevent rapid drainage of dense-graded materials on impervious subgrades. Even after drainage a dense-graded material will hold considerable water by capillarity if protected from evaporation. While open-graded materials will drain more readily, provision must be made to prevent intrusion of fine subgrade soil, and it is difficult to compact them so that traffic will not cause further displacement.

Since both density and drainability are desirable, the range of satisfactory materials is limited, and it is sometimes necessary to choose which property should be given preference.



COEFFICIENT OF PERMEABILITY, $k = \frac{2.3ad}{At} \log \frac{h_1}{h_2}$

Figure 1.—Apparatus for measuring permeability by means of a falling head.

JOHN McADAM, in 1824, wrote that water with alternate freeze and thaw are the evils to be guarded against and, after having secured the soil from under water, the roadmaker should then secure it from rainwater. The paramount importance of drainage with respect to stability of roads is still recognized. However, there is wide difference of opinion as to what constitutes good drainage and how it is to be effected.

This report does not presume to fix design criteria but rather to present some test data on permeability and drainability of soil and to indicate their application to highway subdrainage. Various laboratory and field methods of determining permeability are reported for soils representing the classification groups, for several bituminous paving mixtures, for sieve fractions of sand, concrete sands, and clean aggregates with different minimum sizes, and for graded sand gravels and sands with various amounts and types of material passing the No. 200 sieve. Water held by these latter mixtures after drainage is also reported. The data are used to illustrate methods of calculating vertical capillary flow, flow into horizontal drains, and lateral drainage of base courses. This is followed by a discussion of the interrelation of drainage, density, and gradation of soils as they affect the problems of bearing capacity, intrusion, and pumping.

PERMEABILITY TEST METHODS

For small velocities the rate of flow of water through soil is given by the equation:

$$Q = kAh/d \qquad (1)$$

where

Q=volume of flow per unit time.
k=coefficient of permeability.
A=gross area of soil perpendicular to direction of flow.
h=head loss in a distance d through the soil in the direction of flow.

If a constant head is maintained on a soil sample in a laboratory test, equation 1 may be used to solve for k, thus:

$$k = Qd/Ah \qquad (2)$$

For two layers in series, such as the sample and its pervious support, with thicknesses d_1 and d_2 and permeabilities k_1 and k_2, respectively, the over-all value of k is $(d_1+d_2)/(d_1/k_1+d_2/k_2)$ or, in general,

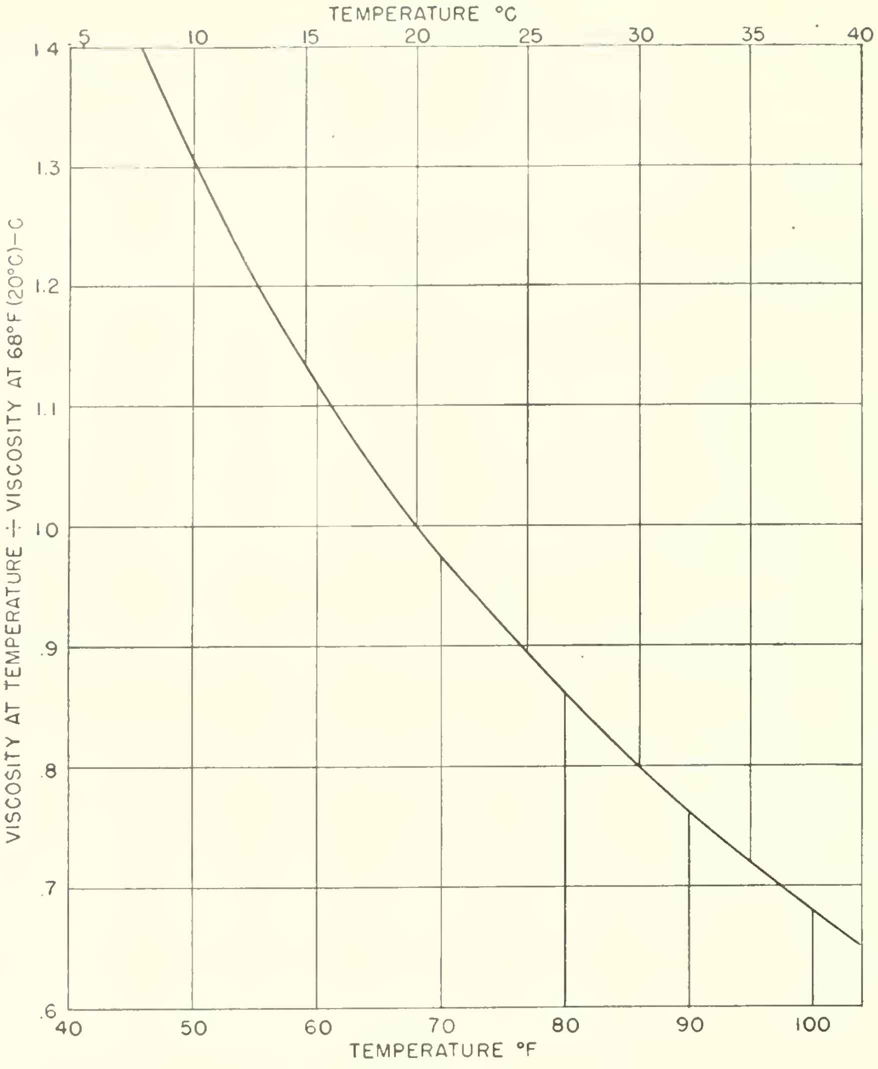


Figure 2.—Effect of temperature of water on the viscosity ratio.

$\Sigma d/\Sigma(d/k)$. If k_2 is large enough, the effect of the support is negligible. For instance, if $k_2=100k_1(d_2/d_1)$, neglect of the support introduces an error of only 1 percent.

In order to facilitate measurement of a small volume Q, a falling head is often used, as illustrated in figure 1. The equation for calculating k, shown in figure 1, is obtained by integration of equation 2, to take care of the fact that the head decreases continuously during the test, and is:

$$k=(2.3ad/At)\ \log\ (h_1/h_2)\ \ldots\ldots\ldots\ (3)$$

The test procedure is as follows: A sample trimmed to size from an undisturbed core, or the desired amount of loose material, is placed in the apparatus. The piston is placed on the soil and loaded or clamped at a given sample thickness. The sample is inundated. The pressure bulb is used to force water from the flask to fill the standpipe from below and thus prevent air from being trapped below the sample. The stopcock is closed when the water is approximately at the initial head. For the less pervious materials, the final adjustment in the initial head is made with the screw clamp. The time required for the water to fall to the final head is recorded.

In a typical example, $a=0.09$ square centimeter, $A=81.08$ square centimeters, $d=1.5$ inches, $h_1=80$ centimeters, and $h_2=40$ centimeters. Then k, from equation 3, $=0.00115/t$ inches per minute, or $0.138/t$ feet per day. The dimension "feet per day" is not the velocity of flow through the soil but a contraction of cubic feet of water per day per square foot of soil for a hydraulic gradient h/d of unity.

The magnitude of the coefficient of permeability may be judged by comparing it with the rate at which water will percolate vertically into a wet soil with a deep water table. For this condition, the hydraulic gradient is unity and the coefficient of permeability is equivalent to the rate of rainfall which could be taken into the soil if the water were uniformly distributed

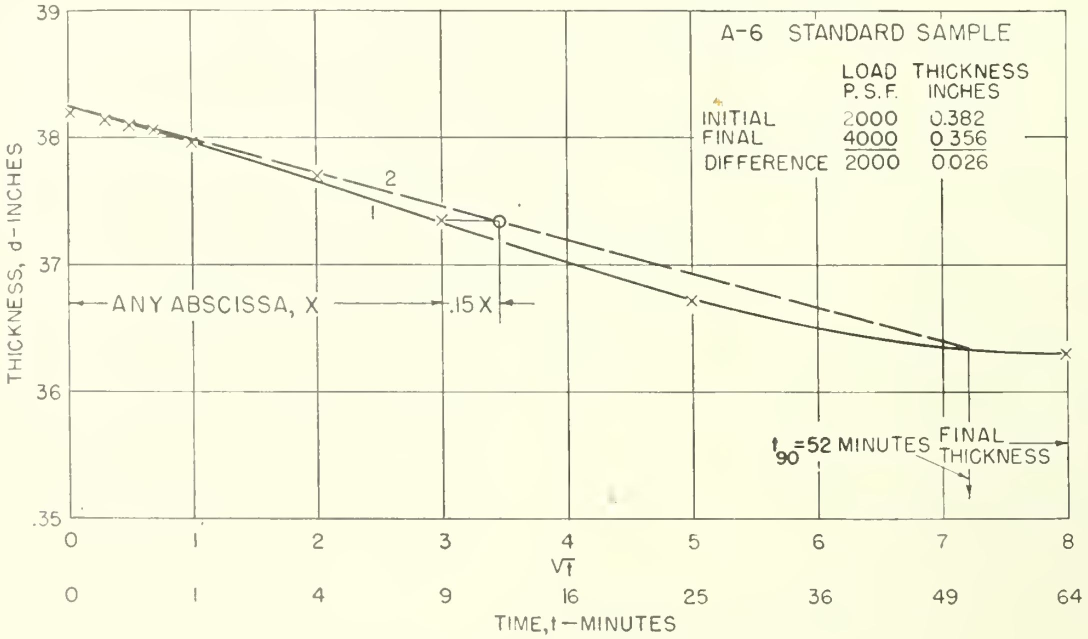


Figure 3.—Thickness change with time.

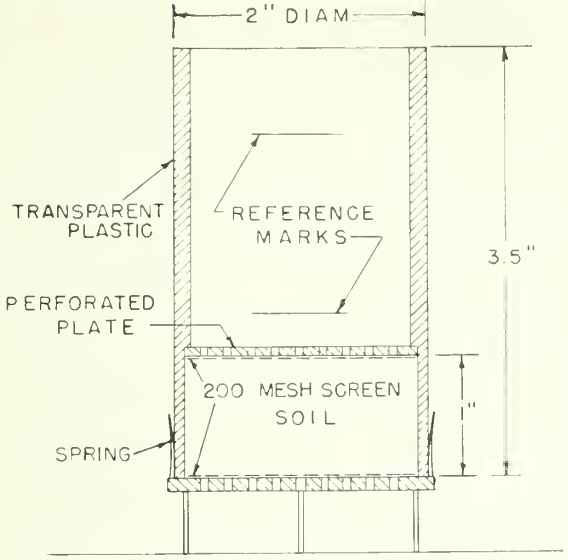
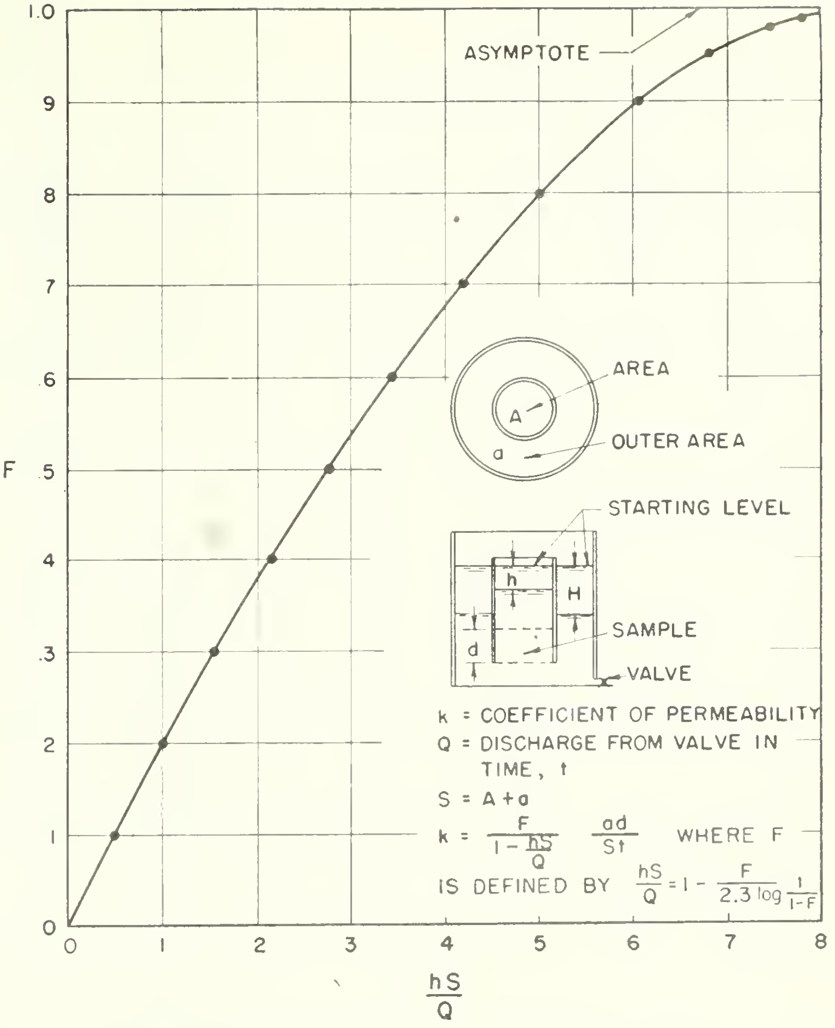


Figure 4.—Permeameter for granular soils.

over the surface of the soil. Thus, a soil with $k=1$ foot per day could transmit vertically downward a maximum rainfall of 12 inches in 24 hours. On the other hand, a soil with $k=0.001$ foot per day would require nearly 3 years to transmit 12 inches of water.

Since the permeability depends upon the viscosity of the water, which is a function of temperature, the calculated permeability at an arbitrary temperature (68° F.= 20° C.) is often reported as $k_{68}=Ck$, where C is the viscosity of water at the test temperature divided by its viscosity at 68° F. Values of C are plotted in figure 2. Ac-



k = COEFFICIENT OF PERMEABILITY

Q = DISCHARGE FROM VALVE IN TIME, t

$S = A + a$

$k = \frac{F}{1-\frac{hS}{Q}} \quad \frac{ad}{St}$ WHERE F IS DEFINED BY $\frac{hS}{Q} = 1 - \frac{F}{2.3 \log \frac{1}{1-F}}$

Figure 6.—Permeability from drainage-lag device.

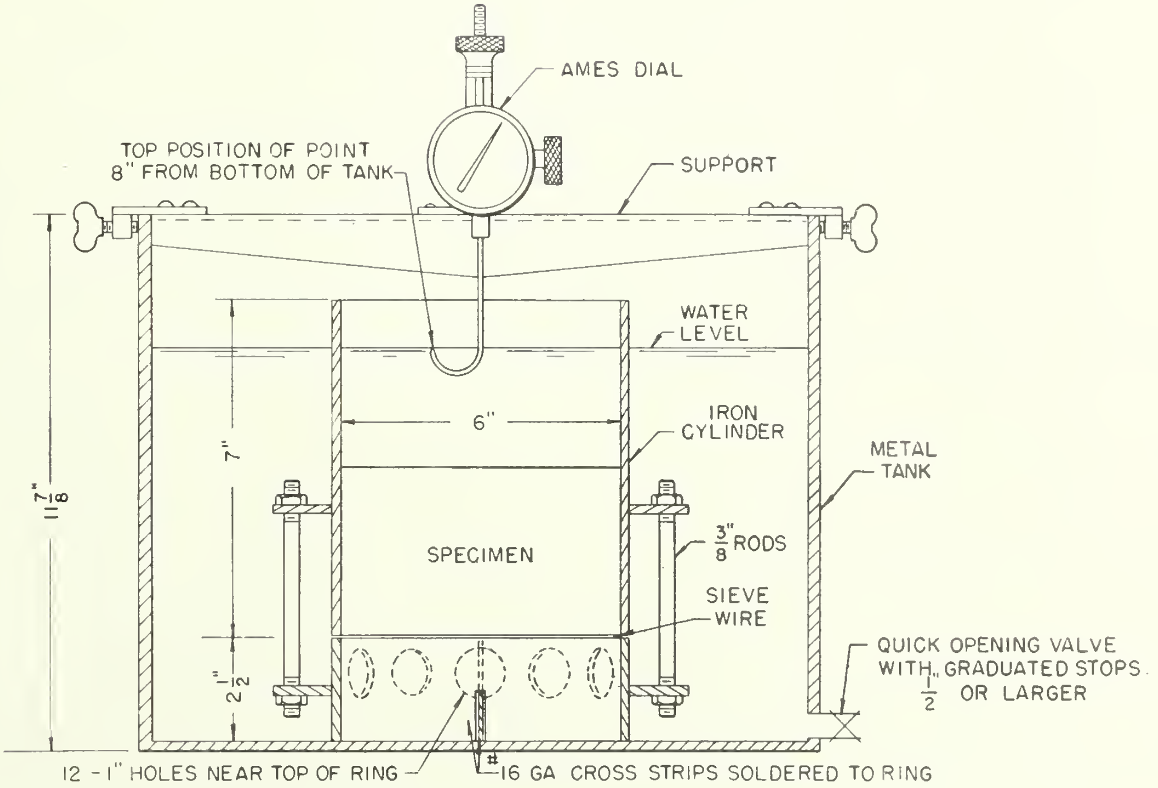

AMES DIAL
TOP POSITION OF POINT 8" FROM BOTTOM OF TANK
SUPPORT
WATER LEVEL
6"
IRON CYLINDER
METAL TANK
7"
11 7/8"
SPECIMEN
3/8" RODS
SIEVE WIRE
2 1/2"
QUICK OPENING VALVE WITH GRADUATED STOPS 1/2" OR LARGER
12 - 1" HOLES NEAR TOP OF RING
#16 GA CROSS STRIPS SOLDERED TO RING

Figure 5.—Drainage-lag permeameter.

tually, the temperature also may affect properties of the soil as well as properties of the water so that it is best to make the test at the temperature of the soil in place.

The Soil Water

The soil water should be used as the permeating liquid since the mineral and gas content of the water is difficult to duplicate. For instance, de-aired water, from which soluble gases have been removed, has sometimes been used for permeability tests. This has been done to prevent air from collecting in the soil and causing reduction in permeability, although this reduction may actually occur under certain field conditions. Similarly, water with either greater or less salt concentration than the soil water may markedly change the permeability of the soil, so that use of distilled water as a standard is not always desirable. The test results reported hereafter were obtained with clean tap water.

The use of the soil water is often impracticable. However, the permeability of saturated fine-grained soils may be determined by laboratory consolidation of undisturbed samples. If the rate at which the thickness of a sample changes after the application of a load increment is observed, the permeability may be computed from an equation based on the theory of consolidation:

$$k=132\frac{d\Delta d}{t_{90}\Delta p}\text{ feet per day} \ldots\ldots\ldots\ldots \quad (4)$$

where

d=initial specimen thickness, in inches.

Δd=reduction in thickness for a load increment Δp, in pounds per square foot.

t_{90}=factor derived from time-thickness relations, in minutes.

To calculate t_{90}, the thickness is plotted against time as in figure 3. The initial portion of the test relation, line 1, is approximately a straight line. Through the intersection of line 1 with the vertical axis, line 2 is drawn with abscissas 0.15 greater than line 1. Line 2 intersects the test relation at t_{90}. Thus, the square root of t_{90}=7.2 gives t_{90}=52. Substituting in equation 4, and using the values shown in figure 3:

$$k=132\frac{0.382\times 0.026}{52\times 2{,}000}=0.000013\text{ foot per day.}$$

This affords an excellent test method for homogeneous materials. Since this test depends upon the rate at which water is squeezed from the soil, the results are not affected to much extent by local conditions, such as root holes, which may be the controlling factor in flow directly through the soil as in the usual permeability test. For two parallel conductors with cross-sectional areas a_1 and a_2 and permeabilities k_1 and k_2, the over-all permeability is $k=(k_1a_1+k_2a_2)/(a_1+a_2)$ or, in general, $k=\Sigma ka/\Sigma a$, so that the more pervious conductor has a dominant influence on the over-all value. Comparison of this relation with that for conductors in series shows that, for stratified deposits, the permeability in the direction of stratification is always greater than that perpendicular to the layers.

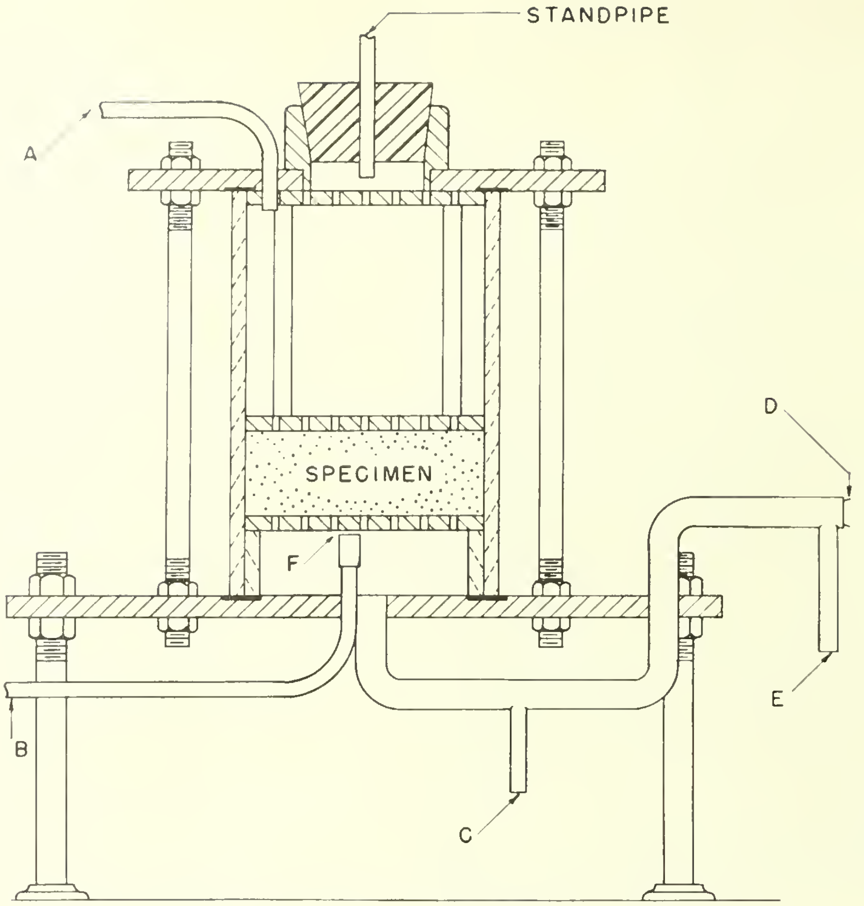


Figure 7.—Permeameter for saturation of sample under vacuum.

Because of the limited permeability of the porous plates in the apparatus shown in figure 1, it is not used for granular soils. Figure 4 shows a device used for sands wherein the sample is retained by 200-mesh screen wire. The same falling-head principle is used but because of the higher permeability the standpipe has a larger area. For field work, a sample may be taken by forcing the device with the base removed into the soil, inverting, and striking off the excess.

To prevent turbulence and minimize migration of particles in testing coarse sands and gravels or base-course mixtures under small gradients, the device shown in figure 5 was developed. The sample may be compacted to any desired density by either an impact or static method. The cylinder containing the compacted sample is inundated in water in the tank. The water level is allowed to come to equilibrium and its level is determined with the hook gage, which is then lowered an arbitrarily selected amount h. The valve at the bottom of the tank is opened and the outflow caught while a stopwatch records the time until the inner water level reaches the hook, at which time the watch is stopped and the valve closed. The effective head is a variable (H-h) where H is the water level lowering outside of the sample. The time, outflow, and lowering of water inside the tube holding the specimen are used in figure 6 with the fixed dimensions of the specimen and apparatus to determine the coefficient of permeability. The formula for k in figure 6 was derived from equation 1 by integration, assuming a constant rate of discharge.

When it is desired to saturate samples under a vacuum before testing, the apparatus shown in figure 7 is used. It may also be used in place of the apparatus shown in figure 1 for materials with relatively high permeability. Water enters at C and air is removed from below the specimen by a tube attached at F to the bottom of the perforated plate which supports a piece of 200-mesh screen wire. When the water reaches the specimen, tube B is closed and water is allowed to pass through and to a height approximately one-half inch above the specimen. C is then closed and water is entered at A until the standpipe is full. The test is performed by allowing water to run out at D or E.

Field Permeability Tests

As shown in figure 8, the coefficient of permeability may be determined on soil

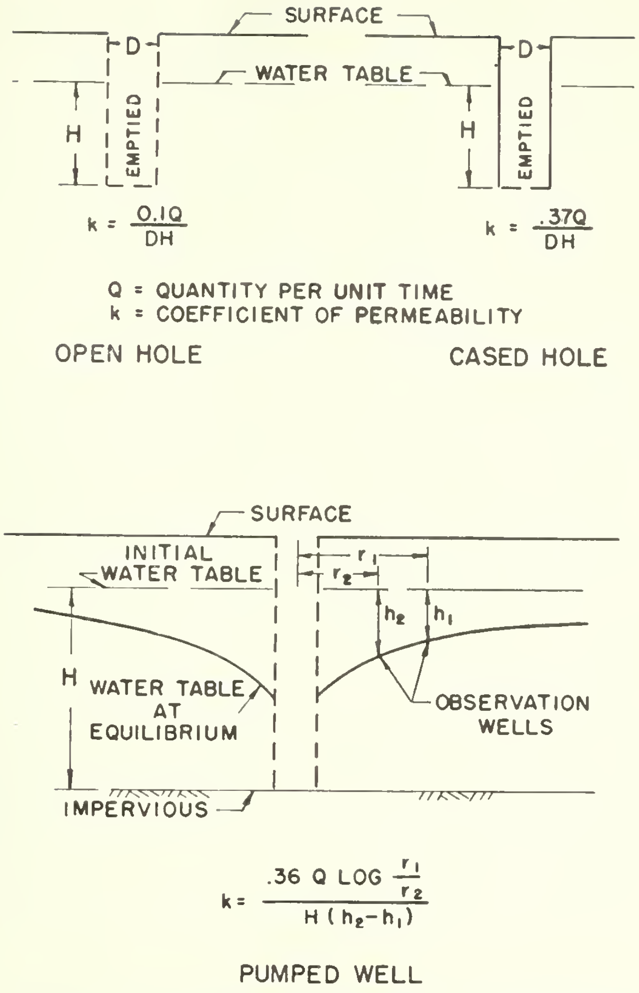


Figure 8.—Determination of coefficient of permeability in the field, below water table.

in place below the water table by drilling a hole in the soil and measuring the water which flows from the soil into the hole (*1, 2*).[1]

The calculated coefficient is an average permeability for soil near the hole, if the soil is not definitely stratified or fissured. The formulas may also be used for water flowing into the soil but there is danger of error in this application because of clogging of the soil surface with suspended particles. If the flow is into unsaturated soil, allowance must be made for capillary forces.

PERMEABILITY TEST RESULTS

Coefficients of permeability for soils of variable grain size and plasticity are shown in table 1. The soil classification is that published by the Bureau of Public Roads in 1942.[2] All of the tests were made on the soil fraction passing the No. 10 sieve. The device shown in figure 1 was used for testing all soils except the A-3 sample, for which the device shown in figure 4 was used. With the exception of the A-3 soil the results shown under the heading "compacted wet" were obtained with test samples prepared by wetting the soil to the liquid limit and compacting it in the cylinder of the device shown in figure 1 under static loads of 1, 2, and 4 kips per square foot. The A-3 soil, a cohesionless sand, was dampened and placed in the cylinder by pressing thin layers firmly into place with a spatula. The test samples for the "compacted dry" condition were molded from air-dry soils under static loads of 1, 2, and 4 kips per square foot, inundated, allowed to drain, and the permeability determined.

The data in table 1 show that for each soil the permeability coefficient (*k* value) decreased with increase in density, and that there is a wide range in the permeabilities of soils of the different groups tested. Also, soils having the same density may have widely different *k* values. For example, test samples in the A-5, A-6, and A-7 groups each having densities of 86 pounds per cubic foot compacted wet had permeability coefficients of 0.015, 0.000009, and 0.0002 foot per day, respectively. Also, while the dry density of the A-3 cohesionless sand and of the A-2 soil compacted wet under 2 kips per square foot was 106 pounds per cubic foot, the permeability coefficient for the sand was approximately 700,000 times greater than that of the A-2 soil.

The relatively high permeability and low density of the micaceous A-5 soil is probably due to the plate-like mica particles that cannot be compacted into a dense structure. The differences in the permeabilities of the same soils compacted by the two different methods described are significant. The results in table 1 show that the samples molded from air-dry materials had *k* values up to 50 times greater than those molded from soils wetted to the liquid limit. These higher values may be attributed to the differences in structure caused by the methods of molding. The soils compacted dry have a less uniform particle arrangement which results in the higher permeabilities.

Permeability of Pavements

Since pavements are placed over soils, their relative permeability is of interest.

Table 1.—***Permeability of soils***

B.P.R. group	Liquid limit	Plasticity index	Amount passing No. 200 sieve	Consolidating load	Compacted wet		Compacted dry	
					Permeability coefficient	Dry density	Permeability coefficient	Dry density
	Percent	*Percent*	*Percent*	*Kips/sq. ft.*	*Ft./day*	*Lb./cu. ft.*	*Ft./day*	*Lb./cu. ft.*
A-1	23	8	26	1	0.00045	114	0.0048	117
				2	.00036	115	.0020	118
				4	.00024	117	.0017	118
A-2	28	11	40	1	.00048	103	.0047	99
				2	.00028	106	.0016	102
				4	.00018	109	.0009	108
A-3	NP	NP	0	(1)	200	106	226	101
A-4	33	12	99	1	.00030	94	.00075	90
				2	....	97	.00060	94
				4	.00022	100	.00024	99
A-5	35	6	35	1	.021	84	.37	78
				2	.015	86	.25	80
				4	.012	90	.17	84
A-6	72	45	86	1	.000035	75	.00011	89
				2	[2].000019	80	.00011	90
				4	[2].000009	86	.00010	92
A-7	67	34	71	1	.00051	77	.026	78
				2	.00023	81	.0084	81
				4	.00020	86	.0012	83
A-8	78	8	38	1	.0039	43	.041	47
				2	.0010	46	.018	48
				4	.00046	49	.010	50

[1] Patted with spatula.
[2] Derived from thickness change with time. Other values from falling-head permeability test.

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[1] Italic numbers in parentheses identify the references in the appendix, page 268.

[2] *Classification of soils and control procedures used in construction of embankments*, by Harold Allen. PUBLIC ROADS, vol. 22, No. 12, Feb. 1942.

Table 2.—*Permeability of bituminous concretes*

Percentage passing No. 200 sieve	Percentage of asphalt cement	Percentage of air voids	Permeability coefficient
Percent	*Percent*	*Percent*	*Ft./day*
5	6	11	0.45
5	6	9.9	.84
7	5.5	9.8	.39
7	6	5.6	.16

The permeability of homogeneous portland cement concrete without cracks or honeycombed structure (*3*) is of the order of magnitude of the permeability of the clay samples A-6 and A-7 in table 1.

The coefficients of permeability for various samples of bituminous concrete compacted in the laboratory under a static load of 3,000 pounds per square inch and then heated to 140° F. for 24 hours were determined as shown in table 2. The coefficient of permeability determined for one sample of bituminous concrete taken from a highway was found to be 0.00002 foot per day. The low permeability found in the field is probably due to traffic compaction, particularly near the surface.

Permeability of Sands

The permeabilities of sieve fractions of Potomac River sand, determined in an apparatus similar to that shown in figure 4, are given in table 3. (The capillary height will be discussed later). Based on published tests of sand under various gradients (*4*), the approximate maximum hydraulic gradient for which equation 1 is applicable is shown in the last column of table 3. Equation 1 is for streamlined or laminar flow. For higher gradients turbulence reduces the flow considerably, so that this gradient becomes a limiting value for testing unless turbulent flow is to be encountered in the field. Ordinarily, for saturated flow, a gradient of one, corresponding to vertical infiltration into a wet soil without ponding, is the maximum encountered in the field. Thus, appreciable turbulence would be found only with materials whose permeability is greater than about 500 feet per day.

Table 4 shows the gradations and permeabilities of three concrete sands whose gradings represent the range of sizes usually permitted in specifications. Their permeabilities correspond approximately to those given in table 3 for the same effective size—that size than which 10 percent by weight is smaller. For instance, with an effective size equal to the No. 80 sieve, the medium sand has a permeability of 113 compared to 118 from table 3 (average for Nos. 60–80 and Nos. 80–100 sieve fractions). While the effective size is useful for sands with a small size range, it is not applicable to materials with a large size range such as the gravels whose permeabilities are given in table 5.

Table 4.—*Permeability of concrete sands*

	Fine sand	Medium sand	Coarse sand
Percentage passing—			
No. 4 sieve	100	100	100
No. 10 sieve	100	82	62
No. 20 sieve	67	49	35
No. 40 sieve	42	28	17
No. 60 sieve	25	15	8
No. 140 sieve	4	0	0
No. 200 sieve	0	0	0
Coefficient of permeability, ft. per day	63	113	194

Table 3.—*Permeability of sieve fractions of sand*

Sand fraction		Saturated capillary height	Permeability coefficient k	Average grain size D	Gradient causing turbulence $300/kD$
Passing sieve No.—	Retained on sieve No.—				
		In.	*Ft./day*	*Mm.*	
10	20	2.5	1.430	1.183	0.2
20	30	3.7	665	.693	.6
30	40	5.2	380	.491	2
40	60	7.9	190	.313	5
60	80	11.7	160	.207	9
80	100	14.0	75	.162	25
100	140	18.5	45	.123	50
140	200	26.4	20	.087	200
200	270	35.6	9	.062	500

Permeability of Graded Aggregates

The material graded from the ¾-inch sieve to the No. 200 sieve, shown as sample 1 in table 5, was designed to represent the middle of the specification of the American Association of State Highway Officials for this fraction of base-course materials. The other gradings were obtained by omitting the fractions below various sieves. The apparatus shown in figure 5 was used for these materials. The wide range of permeabilities is to be noted, as well as the decrease in density as the fines are progressively omitted. The finer fractions have a predominant effect on the permeability, as shown by the increase from 10 to 110 in the *k* values obtained by omitting the 6 percent between the No. 140 and No. 200 sieves from the first mixture.

To determine the effect of material passing the No. 200 sieve on the permeability of aggregate mixtures, several types and quantities of fines were added to Potomac River sand and gravel graded between the ¾-inch and No. 200 sieves. The standard A.A.S.H.O. compaction test was made on each mixture except that the material retained on the No. 4 sieve was not removed.

Table 5.—*Permeability of graded aggregates*

	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
Percentage passing—						
¾-inch sieve	100	100	100	100	100	100
½-inch sieve	85	84	83	81.5	79.5	75
⅜-inch sieve	77.5	76	74	72.5	69.5	63
No. 4 sieve	58.5	56	52.5	49	43.5	32
No. 8 sieve	42.5	39	34	29.5	22	5.8
No. 10 sieve	39	35	30	25	17	0
No. 20 sieve	26.5	22	15.5	9.8	0	0
No. 40 sieve	18.5	13.3	6.3	0	0	0
No. 60 sieve	13.0	7.5	0	0	0	0
No. 140 sieve	6.0	0	0	0	0	0
No. 200 sieve	0	0	0	0	0	0
Dry density, lb. per cu. ft.	121	117	115	111	104	101
Coefficient of permeability, ft. per day	10	110	320	1,000	2,600	3,000

Table 6.—*Gradation of permeability samples for base-course mixtures passing ¾-inch sieve*

Admixture and method of determination	Percentage passing sieve—										
	½-inch	⅜-inch	No. 4	No. 8	No. 10	No. 20	No. 40	No. 60	No. 100	No. 140	No. 200
No admixture:											
Design	86	73	52	37	34	20	12	7	4	2	0
Washed	86	75	54	39	36	23	15	11	7	4	3
Standard		75	56		43		23				6
5-percent admixture:											
Design	86	74	55	40	37	24	17	12	9	7	5
Washed, average	86	75	55	40	38	26	18	13	10	8	6
Standard, average		75	56		41		24				9
10-percent admixture:											
Design	87	75	57	43	40	28	21	17	14	12	10
Washed, average	87	75	57	44	42	29	22	18	14	12	11
Standard, average		75	58		45		28				14
15-percent admixture:											
Design	88	77	59	46	44	32	26	21	18	16	15
Washed, average	88	77	60	46	44	33	26	22	19	17	16
Standard, average		77	60		48		32				18
25-percent admixture:											
Design	89	79	64	52	50	40	34	31	28	26	25
Washed, average	88	79	63	52	50	40	34	30	28	26	25
Standard, average		78	65		53		40				27

For the permeability test, the material was mixed with water to obtain a moisture content 20 percent greater than the optimum (to obtain minimum permeability), tamped to a density slightly below maximum, loaded statically to obtain and maintain maximum density, and saturated from below with tap water. The permeability was determined on duplicate samples in a device similar to that shown in figure 1, except that for permeabilities greater than 0.1 foot per day the apparatus shown in figure 7 was used. After the permeability test, one sample was dried at 110° C., washed on a No. 200 sieve, and the sieve analysis determined. The other sample was air-dried, the sieve analysis determined by the standard A.A.S.H.O. method, and the fraction passing the No. 40 sieve was tested according to standard A.A.S.H.O. procedures to determine the physical characteristics.

Comparison of the average gradations obtained by washing with the design gradations, for each percentage of admixture, as shown in table 6, indicates some degradation due to compaction. Comparison of the gradations obtained by washing with the gradations obtained by the standard method indicates a further degradation due to pulverizing the air-dry material and the mechanical dispersion used for the hydrometer analysis.

Table 7.—*Physical characteristics and permeability of graded aggregate with admixtures of various amounts and types of material passing the No. 200 sieve*

Type and percentage of admixture	Liquid limit	Plasticity index	Compaction test		Molding moisture	Permeability coefficient (68° F.)		
			Maximum density	Optimum moisture		Test 1	Test 2	Average
			Lb./cu. ft.	*Percent*	*Percent*	*Ft./day*	*Ft./day*	*Ft./day*
None	NP	NP	134.5	8.0	9.6	12	13	12
Silica dust:								
5 percent	NP	NP	136	7.0	8.4	.61	.70	.66
10 percent	NP	NP	138	6.2	7.4	.058	.061	.060
15 percent	NP	NP	137	6.6	7.9	.018	.022	.020
25 percent	NP	NP	132	8.2	9.8	.016	.020	.018
100 percent	NP	NP	103	18.3	22.0	.025	.026	.026
Limestone dust:								
5 percent	NP	NP	137	6.8	8.2	.33	.44	.38
10 percent	NP	NP	142	5.6	6.7	.02	.06	.04
15 percent	NP	NP	141	5.6	6.7	.02	.03	.025
25 percent	NP	NP	138	7.2	8.6	.015	.018	.016
100 percent	NP	NP	99	19.6	23.5	.024	.026	.025
Manor loam:								
5 percent	NP	NP	137	7.0	8.4	.08	.11	.10
10 percent	NP	NP	137	6.0	7.2	.02	.02	.02
15 percent	25	4	136	6.5	7.8	.007	.007	.007
25 percent	29	7	132	6.7	8.0	.0026	.0029	.0028
100 percent	38	8	98	22.4	26.9	.0009	.0017	.0013
Keyport silt loam:								
5 percent	16	2	138	6.5	7.8	.043	.067	.055
10 percent	18	6	139	6.1	7.3	.0005	.0015	.0010
15 percent	19	6	140	5.5	6.6	.00009	.00010	.00010
25 percent	21	8	135	6.6	7.9	.00007	.00008	.00008
100 percent	33	14	112	17.0	20.4	.00004	.00004	.00004
Tuxedo clay:								
5 percent	19	5	138	7.0	8.4	.005	.015	.010
10 percent	23	8	137	6.2	7.4	.00030	.00039	.00034
15 percent	26	11	135	5.8	7.0	.00007	.00009	.00008
25 percent	35	17	131	8.0	9.6	.00004	.00006	.00005
100 percent	55	28	106	20.4	24.5	.00002	.00002	.00002

The physical characteristics and permeabilities of the original material and the various mixtures are shown in table 7. It is to be noted that these permeabilities are for samples packed very uniformly in the laboratory. Lack of uniformity in the field may give areas of very high permeability or layers and dams of relatively low permeability.

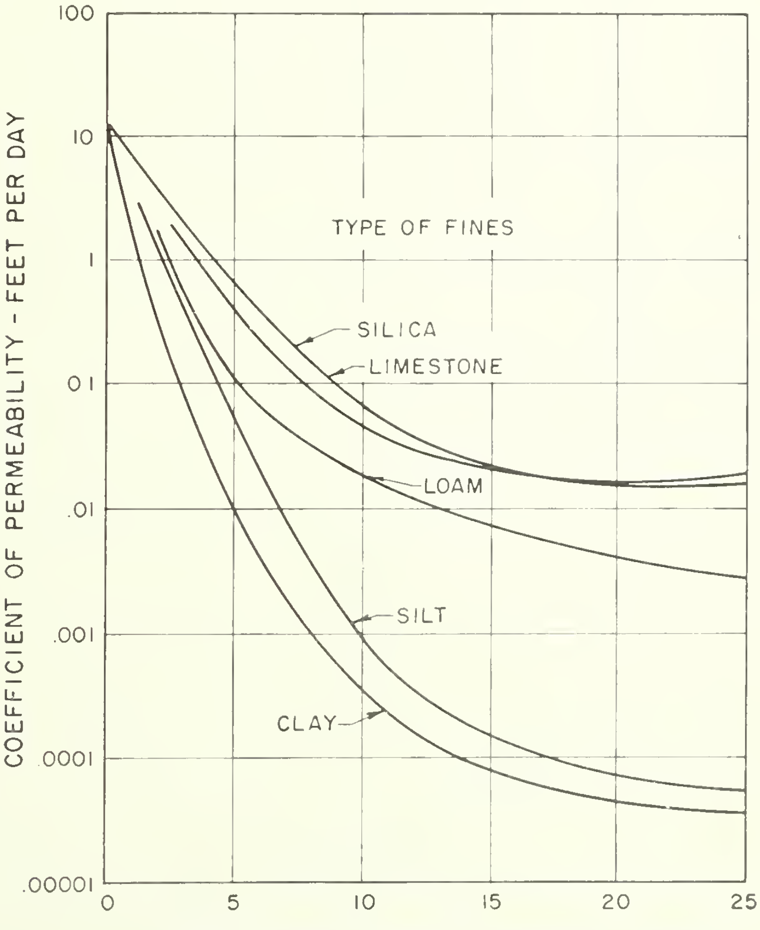


Figure 9.—Effect of fines on permeability of graded aggregate.

The data in table 7 show that a 5-percent admixture causes a marked decrease in permeability, even for the nonplastic admixtures, as illustrated in figure 9. For the same percentage of admixture the permeabilities vary widely with the type of admixtures. With increasing percentages of admixture, the permeabilities approach the values obtained for the admixture alone (100-percent admixture). While the plasticity and gradation both affect the permeability, these values are not sufficient to determine the coefficient of permeability. This is to be expected since the permeability depends upon the arrangement of the particles (structure), which are not considered in the classification tests.

Similar tests were made on graded sand with admixtures of various percentages of fines using the permeameter shown in figure 7. Samples were compacted to maximum density at optimum moisture by the standard A.A.S.H.O. compaction procedure.

The gradation of the mixtures as compounded is shown in table 8, and the compaction and permeability test results are shown in table 9. The results are comparable to those for the gravel-sand fines for the same ratio of fines to sand. Thus the 5-percent admixture in table 7 is roughly comparable to a 9-percent admixture interpolated in table 9, since the sand and fines constitute only 57 percent of the gravel-sand-fines mixture. The permeability of 0.10 for 5-percent manor loam in table 7 is about equal to the value for 9-percent interpolated in table 9. The permeability of the plastic gravel-sand-fines mixtures tends to be somewhat lower due to the fact that the moisture content at the time of

Table 8.—*Gradation of permeability samples for base-course mixtures passing the No. 4 sieve*

Admixture, percent of total weight	Percentage passing sieve—						
	No. 10	No. 20	No. 40	No. 60	No. 100	No. 140	No. 200
0	71	46	30	18	10	4	0
5	73	48	33	22	14	8	5
10	74	51	36	26	19	13	10
15	75	54	40	30	23	18	15
25	78	59	47	39	32	28	25

Table 9.—*Permeability of graded sand passing No. 4 sieve with admixtures of various amounts and types of material passing the No. 200 sieve*

Type and percentage of admixture	Molding		Permeability coefficient (68° F.)		
	Density	Moisture	Test 1	Test 2	Average
	Lb./cu. ft.	*Percent*	*Ft./day*	*Ft./day*	*Ft./day*
None	119.5	11.5	4	5	4.5
Silica dust:					
5 percent	123.0	10.0	.50	.84	.67
10 percent	127.0	9.0	.10	.11	.10
15 percent	126.5	8.5	.047	.050	.048
25 percent	127.5	8.0	.017	.021	.019
Limestone dust:					
5 percent	123.0	7.0	.44	.59	.51
10 percent	129.0	7.0	.07	.14	.10
15 percent	130.5	8.5	.025	.038	.031
25 percent	136.0	8.0	.0048	.0051	.005
Manor loam:					
5 percent	123.5	11.0	.51	.58	.54
10 percent	127.5	10.0	.080	.080	.080
15 percent	128.0	9.0	.044	.037	.040
25 percent	132.0	8.0	.0076	.0030	.0053
Keyport silt loam:					
5 percent	125.5	10.5	.13	.12	.12
10 percent	131.5	8.5	.017	.016	.016
15 percent	133.0	8.0	.024	.029	.026
25 percent	132.0	9.0	.00022	.00024	.00023
Tuxedo clay:					
5 percent	126.0	10.0	.21	.23	.22
10 percent	131.5	8.5	.032	.056	.044
15 percent	131.8	8.4	.019	....	....
25 percent	134.0	8.5	.00015	....	....

compaction was 20 percent above optimum rather than optimum as for the sand-fines.

CAPILLARY STORAGE

In soils which have not been waterproofed to reduce their affinity for water, the surface tension of the water produces an appreciable force which can hold water in the soil above the water table.

The height above the water table to which a soil can stay saturated may be determined by means of the apparatus shown in figure 10. Starting with the water in both tubes at the level of the top of the soil, the stopcock is opened, allowing water to drain slowly from the right-hand tube until the water in this tube rises temporarily when air is first drawn through the soil specimen. The difference between this level and the bottom of the soil specimen is the saturated capillary height shown in table 3 for several sand fractions. For fine-grained soils this test is quite sensitive to changes in density and uniformity of structure since it is a measure of the largest pore.

By using a sealed-in disk, such as unglazed porcelain with a high saturated capillary height, in place of the sieve shown in figure 10, the amount of water held by a soil at equilibrium at various heights can be determined. To increase the effective height of water column, mercury can replace part of the water or a vacuum below or air pressure above may be applied to the specimen. Typical moisture-height curves are shown in figure 11. Since the surface tension is a function of temperature, these curves vary somewhat with temperature—the surface tension decreases about 2 percent for an increase in temperature of 18° F. (10° C.).

At a given height, different materials may be at equilibrium with quite different amounts of water: For instance, at a height of 2 feet, 49 percent of water is held by the clay with the same force that 8 percent of water is held by the sand. Due to hysteresis, depending on whether the material is wetting or drying, a given material may hold different amounts of water at the same height H. Thus, from figure 11, the sand at a height $H=1$ foot may be at equilibrium with moisture contents from 12 to 20 percent at different times depending upon the previous moisture variations.

Specific Yield

The specific yield or volume of water per unit volume of soil removed by drainage is:

$$y=\frac{m_0-m}{100}\times\frac{w_0}{62.4}\ldots\ldots\ldots\ldots\ldots\ldots \qquad (5)$$

where

m_0=moisture content before drainage, in percent.

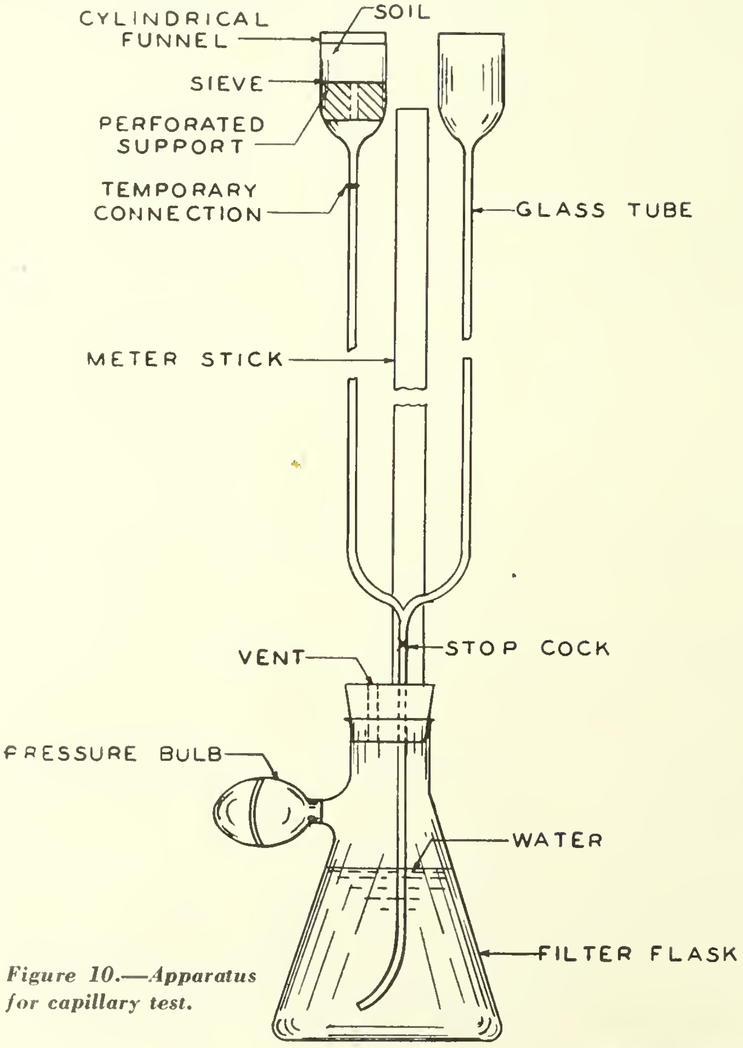


Figure 10.—Apparatus for capillary test.

m=average moisture content after drainage, in percent;

w_0=dry density, in pounds of dry soil per cubic foot of wet soil.

To include water removed from a capillary fringe of height H_1 above the water table when the water table is lowered a depth d, m may be taken as the average moisture held against heights between H_1 and H_1+d.

This is illustrated in figure 12. The initial condition shows that the soil to be drained had a moisture content of m_0 and depth d with a capillary fringe above the water table of depth H_1. The final condition shows the water table lowered a depth d with a capillary fringe of depth H_1+d. The capillary fringe of depth H_1 is common to both conditions so that the moisture drained is from m_0 for the initial condition to the average moisture m in the capillary fringe over the distance d from H_1 to H_1+d above the water table.

For example, consider the sand-clay curve in figure 11, assuming $H_1=1$ and $d=3$. Here $m_0=41$ and the average moisture content from $H_1=1$ to $H_1+d=4$ is $m=29$. Assuming $w_0=90$, then $y=0.17$, from equation 5.

To determine the specific yield of graded sand with various additives passing the No. 200 sieve, the materials represented in table 7 were tamped in lucite tubes of 1¾-inch inside diameter to a depth of 47 inches at optimum moisture and maximum A.A.S.H.O. density. The samples were supported by a piece of 200-mesh screen wire held by a perforated rubber stopper. In some cases, some water came out of both the top and bottom of the tubes after compaction, apparently due to excess pressure built up in the air trapped in the wet soil. After several days a head of water was applied at the bottom of the tubes, creating an upward gradient until water flowed out the top. The purpose was to saturate the samples but, as shown in the first four lines of table 10, the air was not readily displaced and saturation could not be accomplished. Thus, for the material with 5-percent limestone added, the compaction moisture was 7.0 percent. This increased to 10.1 percent before drainage, but 13.4 percent was required for saturation.

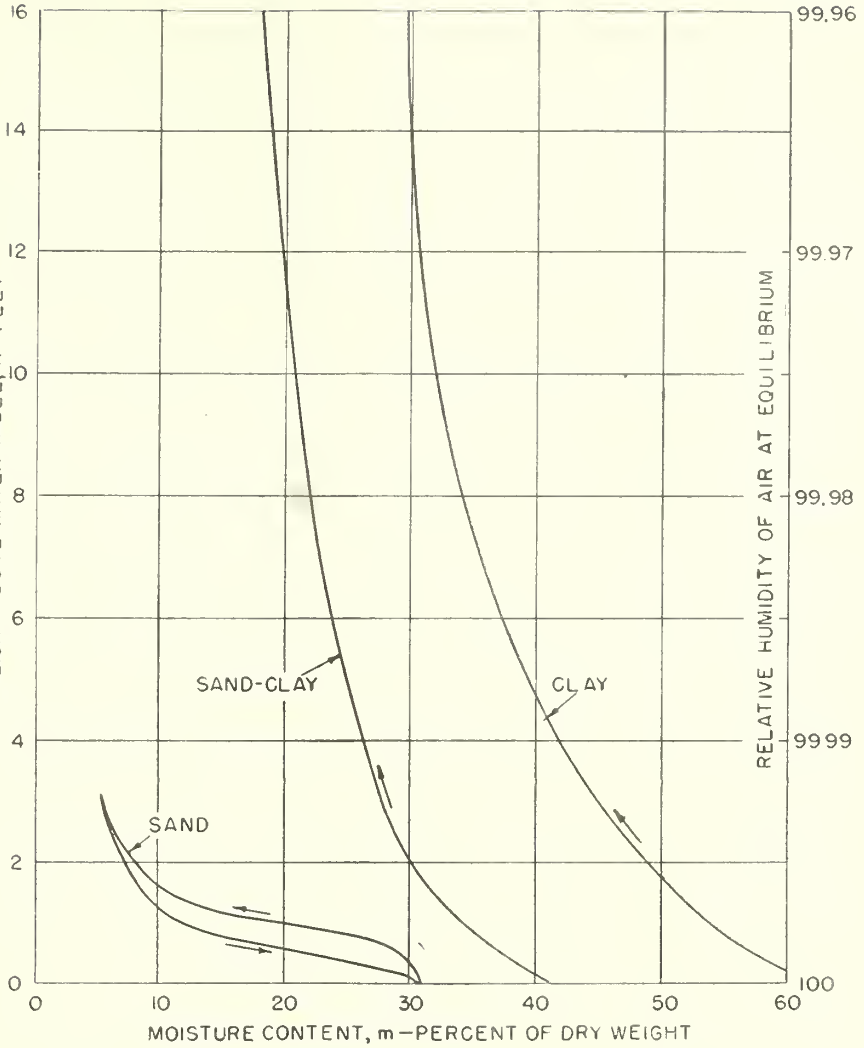


Figure 11.—Moisture-height relations for various soils.

In one case a vacuum was applied at the top, but this resulted in the entrapment of more air rather than less. While greater saturation could have been obtained by evacuating dry material and then admitting water at one end under a vacuum, this would not simulate field conditions where materials are deposited with water at atmospheric pressure. While marine deposits may be saturated, compacted base courses with appreciable fines do not usually become saturated. For example, a base course (with a plasticity index of 3 and with 6 percent passing the No. 200 sieve) with free water above it and resting on a saturated silty clay subgrade was found to have about 5-percent air voids.

After flow through the samples had been established, they were allowed to drain 10 days. The tubes were then emptied in 2-inch increments, starting from the top, and the moisture content of each increment determined. The results are shown in table 10. The moisture contents for

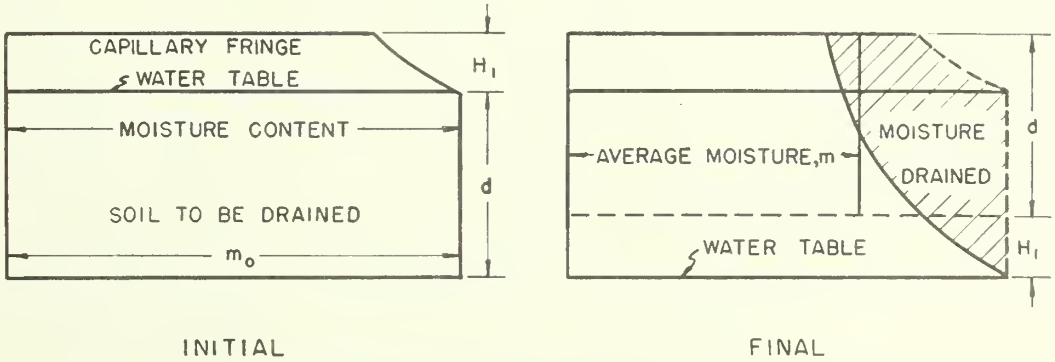


Figure 12.—Specific yield including water from capillary fringe.

Table 10.—Moisture retained after drainage of submerged columns of soil

	Material passing No. 200 sieve added to sand graded from No. 10 to No. 200 sieves[1]									
	Silica		Limestone		Manor loam		Keyport silt loam		Tuxedo clay	
	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent
Dry density, lb./cu. ft. . .	123	127	123	129	123.5	127.5	125.5	131.5	126.0	131.5
Initial moisture, percent. .	10.0	9.0	7.0	7.0	11.0	10.0	10.5	8.5	10.0	8.5
Calculated moisture content, saturation, percent	13.4	11.9	13.4	11.1	13.2	11.6	12.5	10.2	12.3	10.2
Moisture before drainage, percent.	9.2	9.0	10.1	9.2	10.6	9.9	9.8	8.1	9.6	9.4
Moisture at following heights[2] after drainage, percent:										
46 inches.	3.9	6.1	5.1	5.6	5.6	7.7	6.2	6.4	9.5	8.7
44 inches.	3.9	6.2	5.2	6.0	5.6	7.8	6.2	6.4	9.0	8.0
42 inches.	4.1	6.0	5.2	5.8	5.7	7.9	6.6	6.4	8.6	9.0
40 inches.	4.2	6.4	5.4	5.9	6.0	8.2	6.7	6.8	8.7	8.7
38 inches.	4.4	6.4	5.6	6.7	6.1	8.0	6.9	6.7	8.7	8.6
36 inches.	4.4	6.6	5.4	6.0	6.3	8.6	7.0	6.9	8.5	8.1
34 inches.	4.6	6.7	5.4	6.7	6.5	9.2	7.0	7.0	8.8	8.0
32 inches.	4.8	6.8	5.6	6.0	6.6	9.5	7.0	7.1	9.0	8.3
30 inches.	5.3	6.7	5.5	6.0	7.0	9.9	7.1	7.7	9.1	8.8
28 inches.	5.4	6.9	5.7	6.0	7.1	9.8	7.2	8.2	9.3	8.2
26 inches.	5.9	7.1	5.7	6.0	7.4	9.9	7.0	9.0	8.4	9.4
24 inches.	6.1	7.3	5.8	6.0	7.6	10.1	7.4	9.0	8.6	9.5
22 inches.	6.5	7.6	6.0	6.1	7.7	9.7	7.7	9.0	8.2	9.8
20 inches.	7.1	8.0	6.0	6.6	7.8	9.7	8,1	8.9	8.8	9.3
18 inches.	7.8	7.9	6.1	6.1	8.2	10.0	8.1	8.8	8.8	10.0
16 inches.	8.2	8.2	6.2	7.6	8.6	9.7	8.1	8.7	8.6	9.8
14 inches.	8.3	7.7	6.3	7.1	9.1	9.3	8.5	8.4	8.5	9.9
12 inches.	8.1	7.9	6.5	7.1	9.0	8.6	9.0	8.4	8.8	10.0
10 inches.	8.1	7.8	7.2	7.1	8.8	8.6	8.8	8.1	8.9	10.6
8 inches.	7.9	8.0	7.2	6.7	8.9	8.6	8.9	8.0	9.0	9.8
6 inches.	8.2	8.2	7.8	7.0	9.2	8.5	9.2	7.4	9.0	10.0
4 inches.	8.5	8.8	8.5	7.3	9.5	9.1	8.6	7.4	9.2	8.9
2 inches.	9.1	9.2	9.5	8.2	10.1	9.7	8.8	8.4	9.5	9.1
½ inch.	11.0	11.2	11.3	11.0	11.6	11.1	9.9	11.1	10.3	11.3
Average moisture after drainage for 10 days, percent.	6.4	7.4	6.5	7.1	7.7	9.1	7.7	7.8	8.9	9.4
Specific yield:										
Observed.	.055	.033	.071	.043	.057	.016	.042	.006	.014	.000
From saturation to drained moisture.	.138	.092	.136	.083	.109	.051	.096	.051	.069	.017

[1] Physical characteristics of admixtures are shown in table 7.
[2] Center of 2-inch increments except bottom inch.

various heights above the bottom of the tube for the specimens containing 5-percent limestone dust and 5-percent Keyport silt loam are shown graphically in figure 13. Using equation 5, the specific yield was calculated for lowering the water table from the surface to various depths, and the values are shown graphically in figure 14, assuming initial saturation. For material not initially saturated these values should be reduced by the amount of air voids. For the full 47-inch depth, table 10 shows the effect of amount and type of material passing the No. 200 sieve on the specific yield of a well-graded sand.

CAPILLARY FLOW

The curves in figures 11 and 13 are for a condition of static equilibrium—that is, no flow of water. If water is drawn up to a given level in the case shown in figure 13 and removed at a constant rate, as by evaporation or freezing, the moisture content in the soil would decrease until a moisture gradient was established for which this amount of water could be drawn from below the water table. For this condition of dynamic equilibrium or steady state of flow:

$$q=\frac{Q}{A}=k_u\left(\frac{dH}{dz}-1\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

where

q=rate of removal of water, in depth (volume per unit area) per unit time.

k_u=coefficient of unsaturated permeability, a function of H.

H=height above water table for static equilibrium corresponding to the moisture content.

z=vertical dimension, positive upward.

The relation between k_u and H has been determined in various ways, such as with permeameters like that shown in figure 7, using negative heads or by measurement of tension gradients in columns of soil subjected to evaporation. Constant temperature is essential because a small change in the moisture content of the soil may account for a large percentage of a small flow. One method of determining the tension in the soil moisture is to measure the electrical resistance of a cell buried in the soil. The cell consists of two electrodes separated by an inert porous medium, and is first calibrated under known moisture tensions.

The form of the empirical relation between k_u and H can be approximated over a limited range by the equation: $\log k_u = \log a - H/b$, where a and b are empirical constants.

For the range $H=1$ to $H=10$ feet, typical values of a, in feet per day, and b, in feet, as determined by R. E. Moore (5), are:

	a	b
Sand	0.28	0.66
Silt	.045	3.9
Light clay	.005	7.5

Substituting the general expression for k_u in equation 6, and integrating, gives:

$$q=a\,\frac{1-\text{antilog}\,\dfrac{z-z_o+H_o-H}{b}}{\left(\text{antilog}\,\dfrac{z-z_o}{b}-1\right)\ \text{antilog}\,\dfrac{H_o}{b}} \qquad (7)$$

For a layer in contact with the water table ($z_o=H_o=0$), the maximum capillary flow upward (infinite H) to a height z is:

$$q=\frac{a}{\text{antilog}\,\dfrac{z}{b}-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

or the maximum height for a given rate of flow is:

$$z=b\ \log\left(\frac{a}{q}+1\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

With the values of a and b noted above, this equation gives the values plotted in figure 15 which shows that, while the clay can lift 1 inch of water per year to the greatest height, the silt gives the maximum height for larger rates comparable to those required for appreciable frost heave. Rates of flow due to capillarity may be much greater than those due to gravity alone. For instance, while the clay (fig. 15) can lift 100 inches of water per year 0.6 foot, the maximum rate of gravity flow (unit hydraulic gradient) is only 0.005 foot per day (k_u for $H=0$ which equals a) or 22 inches per year. Similar effects have been noted with portland cement concrete for which the water transmitted by capillarity is much greater than that forced through a sound sample by ordinary pressures.

Effect of Layers

The effect of a layer of sand on the capillary rise of water is shown in figure 16. (Figs. 16 and 17 show soil cross sections and corresponding moisture content curves). With an impervious surface, so that there is no flow through the surface, as shown on the left side of figure 16, each material attains the moisture it would hold at a given height if it were continuous to the water table—that is, the sand does not affect the moisture in the silt at static equilibrium. If water evaporates continuously from the surface, as at the right in figure 16, the soil will dry out enough to establish a tension gradient sufficient to maintain the flow required for continuous operation. The sand causes more drying of the top layer than would occur without it because the unsaturated permeability of the sand at appreciable height above the

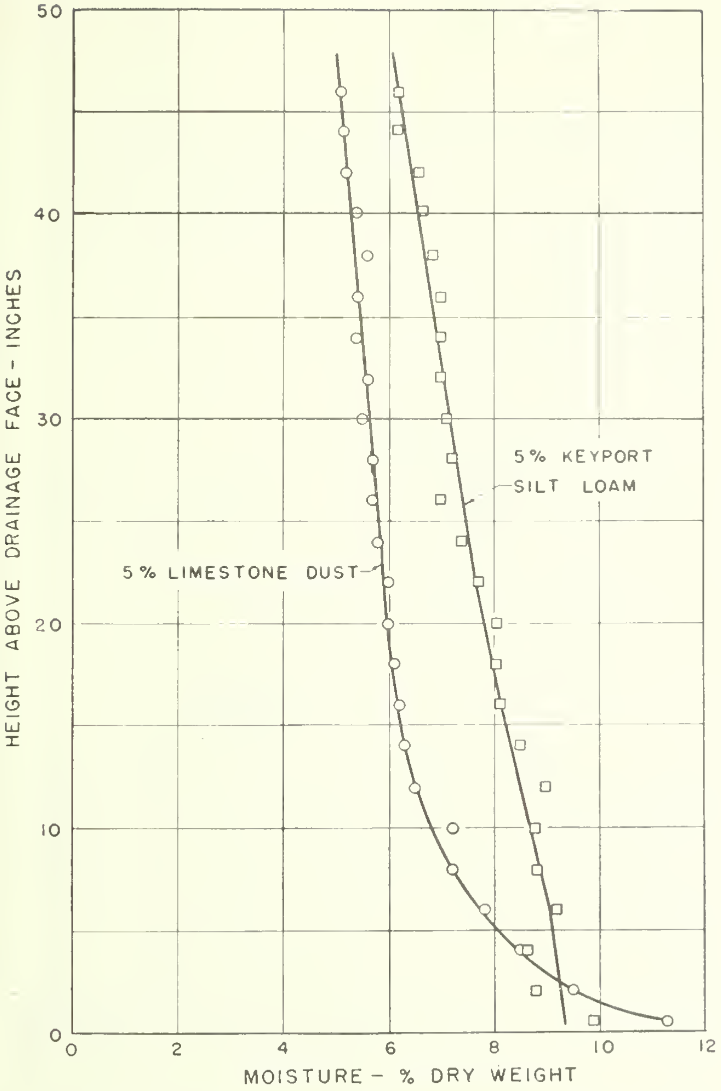


Figure 13.—Capillary retention in drained columns of soil.

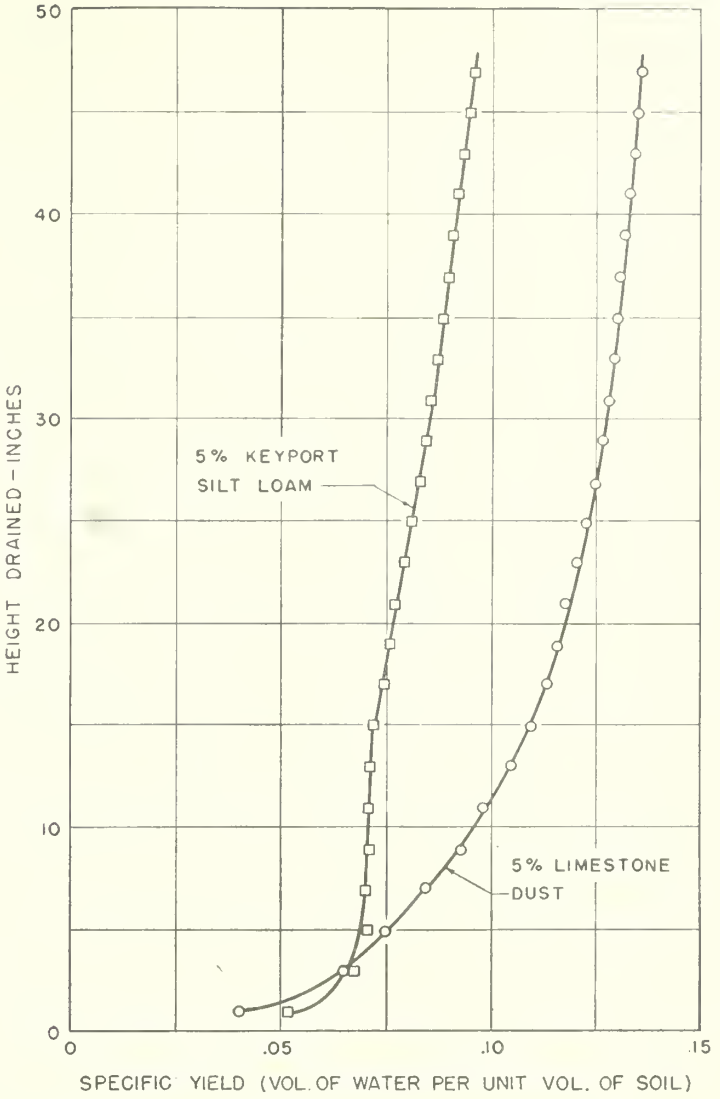


Figure 14.—Specific yield for various thicknesses, assuming initial saturation.

water table is less than that of silt at this height. If evaporation stops, the moisture increases toward the values for static equilibrium. Upward flow may be satisfactorily limited by a granular layer where there is appreciable evaporation. A buried impervious layer, such as bitumen, above the water table has also been used.

As shown in figure 17, if water is supplied at the surface, the moisture content is temporarily higher in a silt layer than for conditions of static equilibrium. The presence of a sand layer at appreciable height above the water table retards the downward flow due to its relatively low unsaturated permeability at this height (*6*). For example, a mass of soil in a lysimeter, a box with a perforated bottom separated from the drained undersoil to permit weighing and measurement of percolation, is found to be wetter than the surrounding soil unless a vacuum is applied to its base to replace the moisture tension which the undersoil ordinarily supplies. Thus, for a considerable time after rain ceases the upper layer of silt will be wetter than it would have been without the sand layer. If the water table is kept

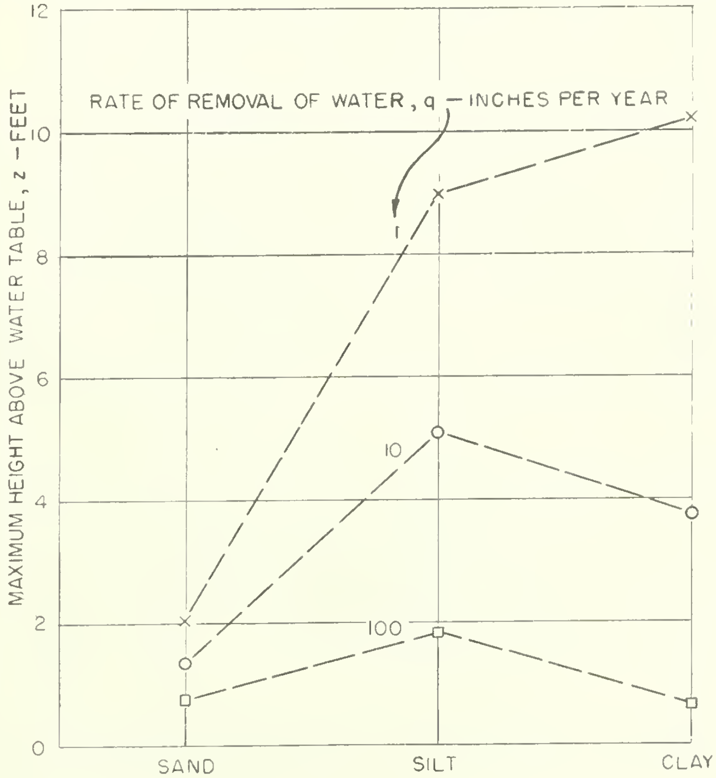


Figure 15.—Calculated rates of capillary flow.

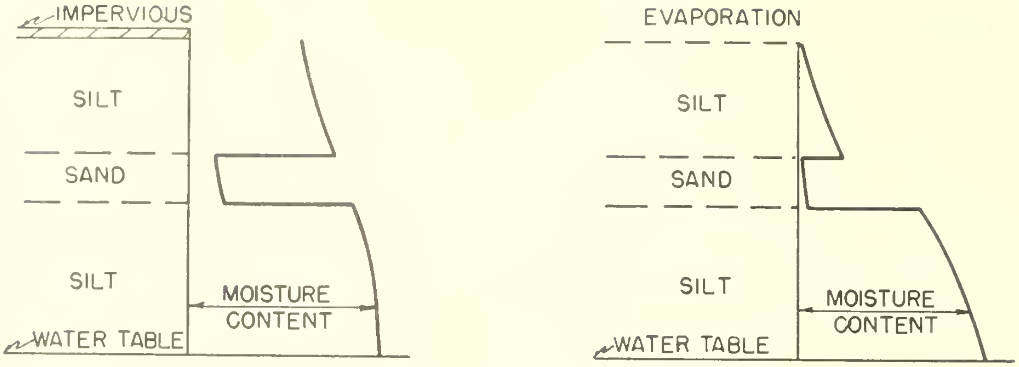


Figure 16 (left).—Capillary rise of water.

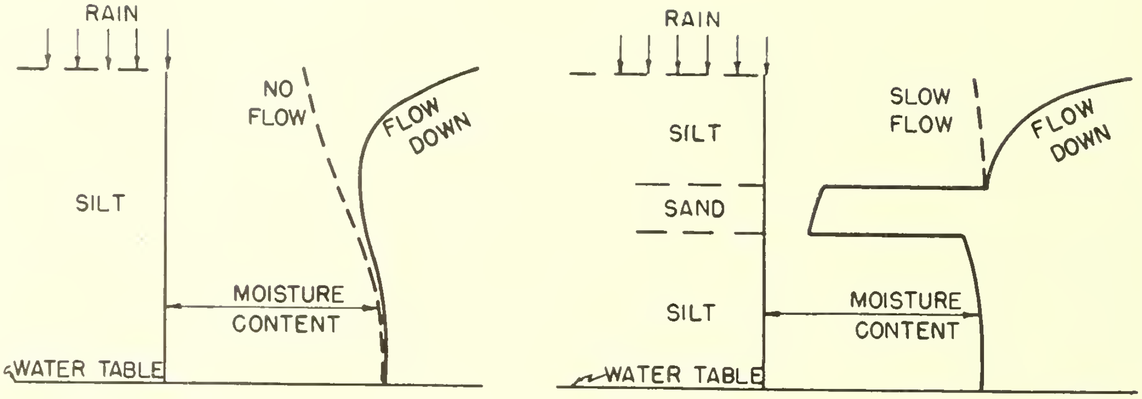


Figure 17 (right).—Downward capillary flow.

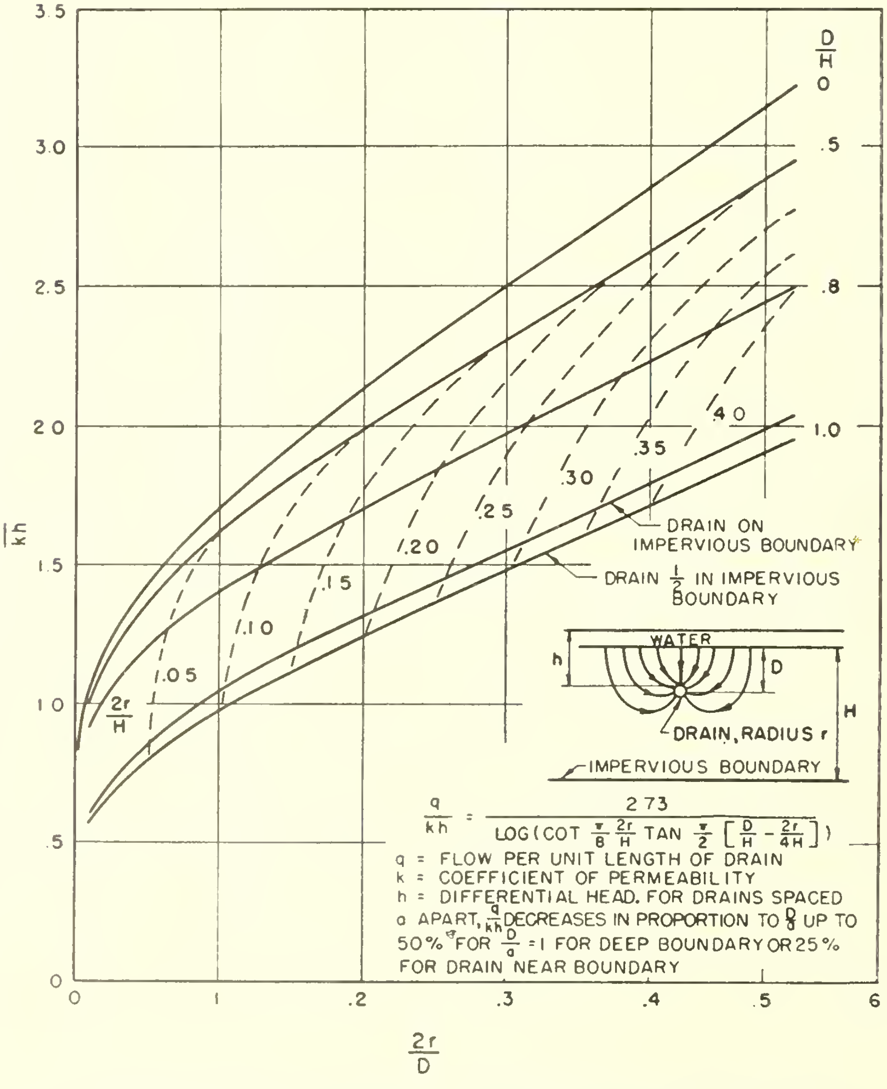


Figure 18.—Flow into buried horizontal drain from flooded surface

low in a subgrade, a base on the subgrade may drain better than one placed on a highly porous subbase.

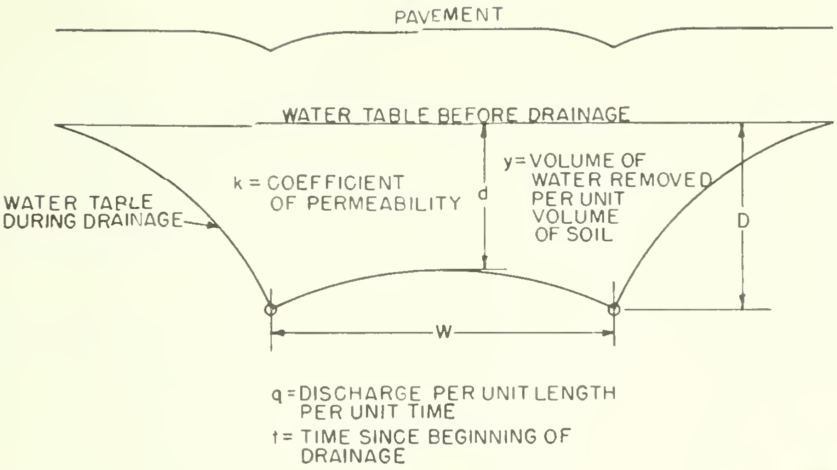
Other Influences

Temperature difference in the soil causes flow of water since, for equal initial moisture content, water tends to move from a warmer to a cooler area. Another force which causes water to flow is the osmotic pressure such as is caused by differences in concentration of salts at different locations in the soil profile. Flow of soil water may also be caused by an electrical potential. Thus, if two electrodes are placed in a soil and a direct current passed between them, water will flow from the vicinity of the positive electrode to the negative electrode, which could be constructed as a drain. The high cost of the reported field applications which have been made may possibly be overcome by a comprehensive study of this method.

Water may also move through a soil as a vapor. If the air moves as a body, considerable water may be transferred by convection. If the air is still, the vapor may move by diffusion but this is very slow. Since the soil air is generally so nearly saturated, a small decrease in temperature will cause condensation. In some arid regions water has accumulated under pavements, apparently from condensation associated with the rapid cooling of the surface due to radiation under clear skies.

HORIZONTAL PIPE DRAINS

Figure 18 shows the flow of water into buried horizontal drains from a flooded surface, as derived by Kirkham (7). The


PAVEMENT
WATER TABLE BEFORE DRAINAGE
WATER TABLE DURING DRAINAGE
k = COEFFICIENT OF PERMEABILITY
d
y = VOLUME OF WATER REMOVED PER UNIT VOLUME OF SOIL
D
W
q = DISCHARGE PER UNIT LENGTH PER UNIT TIME
t = TIME SINCE BEGINNING OF DRAINAGE

DIMENSIONLESS RATIOS		
$\frac{tkD}{yW^2}$	$\frac{d}{D}$	$\frac{q}{kD}$
0.001	0.06	0.80
0.01	0.37	0.47
0.1	0.79	0.25

Figure 19.—Drainage by two parallel horizontal pipes.

water on the surface could be a film due to rain or water in a permeable base on a much less permeable subgrade. The type of drain is not considered; it is assumed that there is negligible resistance to water entering the drain.

For example, for 6-inch drains, in lines spaced $a=10$ feet apart, resting on an impervious boundary at a depth $H=5$ feet: $2r/D=0.5/4.75=0.105$, and figure 18 gives $q/kh=1.05$. For a drain near the impervious boundary, this is reduced by D/a times 25 percent of its value. Thus, the adjusted $q/kh=1.05-(4.75/10)\times 0.25\times 1.05=0.93$. For $h=4.5$, $q=0.93\times 4.5k=4.2k$. For $k=0.1$ foot per day, $q=0.42$ cubic foot per day for each foot of length. This is equivalent to $(0.42/10\times 1)\times 12=0.50$ inch per day average infiltration through the surface.

Figure 18 may also be used for computing drainage of a pervious substratum under artesian pressure by inverting the defining sketch shown in the figure. Figure 18 is for a steady state where the flow is continuous with time.

For the unsteady state where the water table is lowering, figure 19 shows the drainage of a pavement foundation by two small parallel horizontal pipes in the upper part of a deep soil, as determined by McClelland (*8*). Experimentally determined relations between several dimensionless ratios are shown in figure 19. These ratios may be used to solve various problems, depending upon which values are known or assumed. As an example, take $W=30$ feet, $D=3$ feet, $y=0.1$, and $k=0.25$ foot per day. Suppose the time and rate of flow are desired when d/D reaches 0.79. Then d in figure 19 is $0.79\times 3=2.4$ feet; $tkD/yW^2=0.1$, so that $t=(0.1\times 0.1\times 30\times 30)/(0.25\times 3)=12$ days; and $q/kD=0.25$, giving $q=0.25\times 0.25\times 3=0.19$ cubic foot per day per foot. This discharge rate is also the maximum rate of infiltration for which the drains could maintain the drained depth d at 2.4 feet. For $k=0.0025$ and the same drained depth, q becomes 0.0019 and t becomes 1,200 days, or, for the same time, $t=12$ days, d/D is 0.06, and d is only 0.18 foot.

BASE COURSE DRAINAGE

If a base course were placed over a relatively impervious subgrade, lateral drainage would be required to drain water entering through the surface. The permeabilities of some bituminous surface mixtures as compacted in the laboratory, previously presented, are higher than those of base course mixtures with an appreciable amount of fines. The effect of traffic and cracks on infiltration needs to be determined. If frost penetrates into the subgrade below a base course, the base is apt to become saturated when thaw occurs from the surface. The frozen subgrade, even though permeable when thawed, may prevent drainage downward so that the water in the base may have to drain laterally to escape.

For a base course on an impervious subgrade which is flooded and then allowed to drain along one edge, the rate of drainage may be approximated (*9*) by the formula:

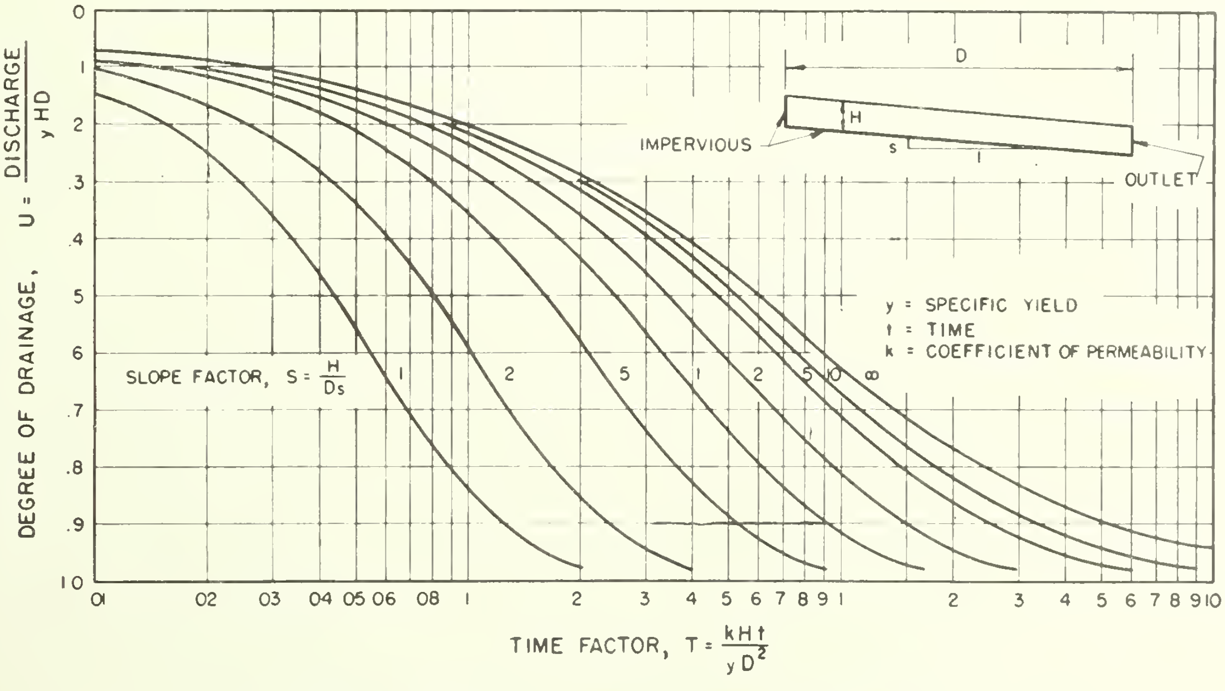


Figure 20.—Rate of drainage of flooded base course.

$$T=US-0.48S^2 \log (1+4.8U/S) \ldots (10a)$$
for $0 \leqq U \leqq 0.5$, and

$$T=0.5S-0.48S^2 \log (1+2.4/S)+1.15S \log (S-US+1.2)/(1-U) (S+2.4) \ldots (10b)$$
for $0.5 \leqq U \leqq 1$

where

$T=kHt/yD^2$=time factor.

k=coefficient of permeability, in feet per day.

H=depth of base, in feet.

t=time, in days.

y=specific yield.

D=width of base, in feet.

$S=H/Ds$=slope factor.

s=cross slope (as 1 percent=0.01).

U=degree of drainage (as 1 percent=0.01).

For horizontal base (S=infinity) the equations are:

$$\left.\begin{array}{l} T=2.4U^2 \text{ for } 0 \leqq U \leqq 0.5 \\ T=0.6U/(1-U) \text{ for } 0.5 \leqq U \leqq 1 \end{array}\right\} \ldots (11)$$

Curves derived by substituting values in formulas 10 are plotted in figure 20. For U=50 percent, this relation may be closely approximated by:

$$\left.\begin{array}{l} T=0.44/(0.74+1/S) \text{ or} \\ t=(y/k) \times 0.44D^2/(0.74H+Ds) \end{array}\right\} \ldots (12)$$

For example, consider a base course 0.5 foot thick and 20 feet wide with 1-percent cross slope, composed of graded sand with 5-percent limestone dust passing the No. 200 sieve (such as shown in tables 9 and 10) and placed on an impervious subgrade. For this material k=0.51 foot per day and, from figure 14, y=0.05 between a height of 0 and $H+Ds$ (the lowering of the water table) $=0.5+20 \times 0.01=0.7$ foot or 8.4 inches. Assuming, for example, 4-percent air voids before drainage, the time required for 50-percent drainage, from equation 12, is $t=(0.05/0.51) \times (0.44 \times 20^2)/(0.74 \times 0.5 + 20 \times 0.01)=30$ days.

The results of similar calculations, using figure 20, for other types of fines and for various slopes, thicknesses, and widths are shown in table 11, assuming initial saturation and final moisture as shown in table 10. The relatively short times required for drainage of the mixtures of sand with Tuxedo clay are due to the small yield which means that, even after 50 percent of the drainable water is gone, they are still almost saturated. Comparing these values with the empirical requirement of 50-percent drainage in 10 days suggested by the Corps of Engineers (*10*) indicates that, according to this proposed criterion, graded base courses with as little as 5 percent passing the No. 200 sieve will not drain satisfactorily except for narrow widths. Less densely graded or stratified material may give higher permeabilities, and therefore quicker drainage, even though appreciable material passes the No. 200 sieve. If the materials do not become saturated, it may be that rapid drainage is not necessary. In any event, materials with low plasticity and more than 5 percent passing the No. 200 sieve have often been satisfactory as highway base courses despite slow drainage, even when subjected to frost.

Neglecting entrance losses, the quantity of water transmitted by steady flow through a sloping base course on an impervious subgrade (*9*) is:

$$q=\frac{kH(sD+H/2)}{D} \ldots \ldots \ldots \ldots \ldots (13)$$

This corresponds to the rate of flow for 42-percent drainage starting from a saturated condition. Thus, roughly, for infiltration through a joint the base would become only 58-percent saturated if drainage were provided at the side. For example, assume a 2-percent slope, H=0.5 foot, D=20 feet, and k=1 foot per day. Then $q=[1 \times 0.5(0.4+0.25)]/20=0.016$ cubic foot per day as the maximum quantity that could be transmitted continuously from a joint or crack, any available excess being forced to run off. Over an area of 20×1 foot, this quantity of water could be transmitted vertically into a drained subgrade if its permeability were as much as 0.0008 foot per day. If the base had a k value of 100 feet per day, it could transmit 1.6 cubic feet per day which would require a subgrade permeability of 0.08 foot per day for continuous infiltration. While a dense-graded base of low permeability will not transmit water as fast as an open-graded base, it will not have as much total capacity for water, must lose much less water for a given percentage of drainage from a flooded condition, and may make much less water available to the subgrade by its lower infiltration capacity.

Table 11.—*Time required for lateral drainage of 50 percent of drainable water for saturated base course on impervious subgrade*

Slope, thickness, and width of base course	Time required for drainage with following material added to sand graded from No. 10 to No. 200 sieves									
	Silica dust		Limestone dust		Manor loam		Keyport silt loam		Tuxedo clay	
	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent	5 percent	10 percent
	Days	*Days*	*Days*	*Days*	*Days*	*Days*	*Days*	*Days*	*Days*	*Days*
1-percent slope, 6-inch thickness, and width of—										
5 feet	4	16	5	24	3	16	16	69	8	6
10 feet	12	54	16	81	11	54	54	230	25	20
15 feet	25	112	35	130	23	108	108	460	49	41
20 feet	42	185	56	215	38	178	180	760	80	67
25 feet	60	270	82	300	55	255	255	1,090	117	97
30 feet	82	370	114	415	76	345	345	1,475	160	130
1-percent slope, 12-inch thickness, and width of—										
5 feet	2	9	3	10	2	8	8	36	4	3
10 feet	8	37	11	40	8	32	32	140	15	12
15 feet	16	78	24	84	16	67	68	290	32	26
20 feet	26	126	39	135	26	108	109	460	53	41
25 feet	39	190	57	200	37	160	160	675	80	60
30 feet	53	260	80	275	52	215	220	920	110	82
2-percent slope, 6-inch thickness, and width of—										
5 feet	3	13	4	15	3	14	13	58	6	5
10 feet	10	45	14	52	9	45	44	190	20	17
15 feet	20	94	28	103	20	86	86	370	40	33
20 feet	32	150	45	165	30	135	135	575	63	51
25 feet	45	205	63	225	42	185	185	785	88	70
30 feet	61	270	84	300	55	240	240	1,015	115	90
2-percent slope, 12-inch thickness, and width of—										
5 feet	2	9	3	10	2	8	8	36	4	3
10 feet	6	31	10	34	6	27	27	115	14	10
15 feet	13	65	20	68	13	54	56	230	27	20
20 feet	22	107	33	114	22	89	93	380	46	34
25 feet	32	155	49	165	31	130	135	545	66	48
30 feet	44	210	70	230	42	175	185	740	90	65

WATER AND STRENGTH

The presence of water in a base material may decrease its strength in several ways. It reduces the cohesion by lowering the capillary forces; it reduces the friction by reducing the effective weight of the material below the water table; and, for quickly applied loads, it may reduce the strength by the development of pore pressure.

Tests made on sands by other investigators (*11*) showed the bearing capacity to be decreased more than 50 percent due to complete submergence as compared to dry sand. Capillary saturation gave somewhat less reduction. Under dynamic loads even greater loss in strength was obtained. The effects of wetting were especially noticeable for low initial densities of the sand.

Rise of water table in the base also affects the strength of subgrade by reducing the effective pressure on the subgrade. For example, if a 12-inch base and surface course weighing 140 pounds per cubic foot is submerged, the effective weight is reduced to 140−62.4=77.6 pounds per cubic foot and the effective pressure on the subgrade is reduced from 140 to 77.6 pounds per square foot. According to figure 21, which shows the strength of a clay at equilibrium under various surcharges, the compressive strength of the subgrade may be reduced from 1.4 to 1.0 kips per square foot by the submergence of the base.

The strength of granular materials under quickly applied loads is greatly affected by density, especially if saturated. This is because the strength depends on the effective stress, which is the total stress minus stress on water or pore pressure, and the pore pressure depends on the density. Thus, if a loose saturated granular material is distorted by shear stresses, the tendency to become denser causes pore pressure (as shown in the upper part of figure 22), which reduces the effective normal stresses and thereby reduces the strength. On the other hand, a dense material tends to expand when sheared (as shown in the lower part of figure 22) so that no pore pressure is developed and the total pressure is effective in developing shearing resistance through internal friction.

The fact that some bases appear stable when saturated may be due to the fact that they are dense enough so that they must expand to shear even after a loss of density due to freezing or other causes.

For materials near saturation, freezing may cause a decrease in density due to the expansion of water in freezing, even without ice lenses. Thus, 20-percent water by volume upon freezing increases to 22 percent, causing a 2-percent decrease in density. It has been suggested by C. H. McDonald (*12*) that materials which do not shove during compaction (displace vertically upward at high moisture contents) will be stable when properly compacted regardless of wet and freezing conditions.

While high permeabilities may be obtained by using coarse aggregates, care must be taken to prevent a reduction in their permeability and stability by intrusion of finer soils to which they may be placed adjacent. For instance, if two layers of material are used to make up the test sample in the apparatus shown in figure 23, and a repetitive load similar to traffic loading is applied by means of the motor and cam, the finer material will be intruded into the pores of the coarser material if the difference in size is too great. Thus, many macadam roads placed on clay have failed when the clay became wet and soft, as indicated in the sketches on the left side of figure 24. Well-graded aggregate or a fine aggregate subbase, as shown on the right side of figure 24, will prevent intrusion.

Use of Filters

Subsurface drainage of soil in cut slopes and under pavements is often accomplished by the use of drain tile or perforated pipe placed in a trench and covered with a granular material. The granular material is commonly designated as the filter. If the voids of the filter material are very much larger than the finer grains of the soil to be drained, the fine soil particles are likely to be washed into the pipe or into the interstices of the filter (left and center sketches, fig. 25), where they accumulate and gradually obstruct the flow.

When the finer particles of the filter material at or near the plane of contact with

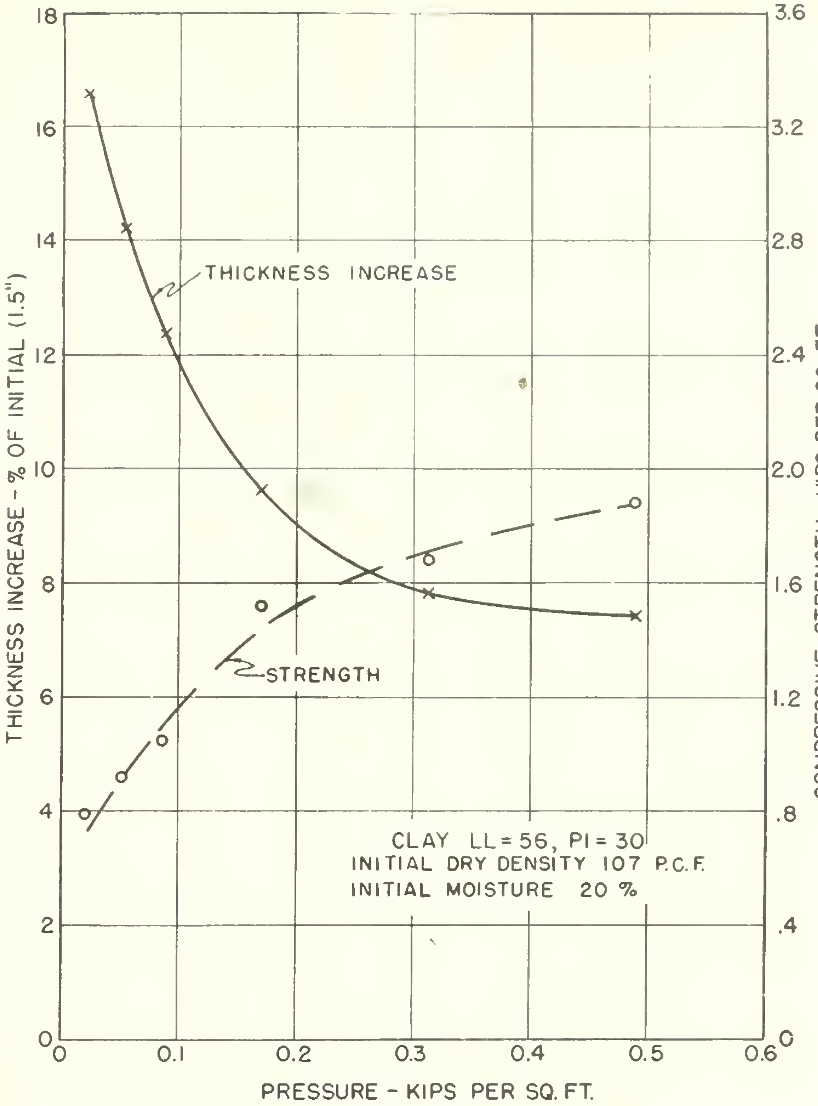


Figure 21.—Strength of clay immersed under various surcharges.

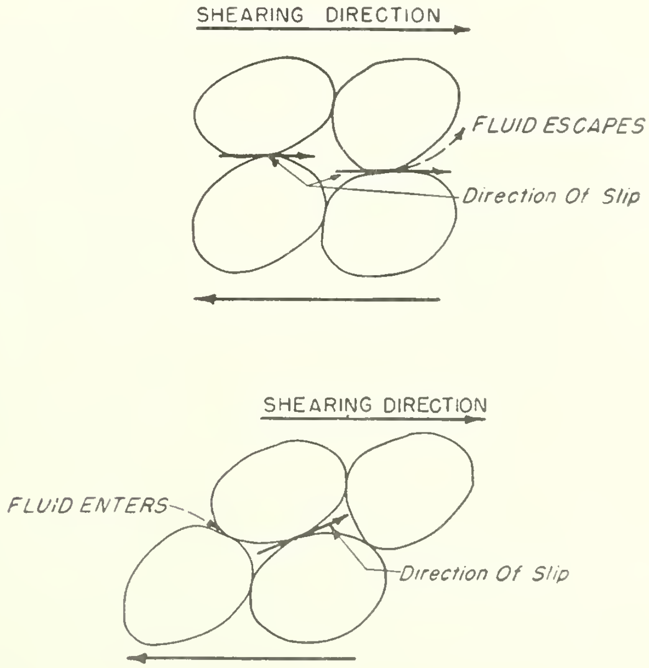


Figure 22.—Volume change with shearing.

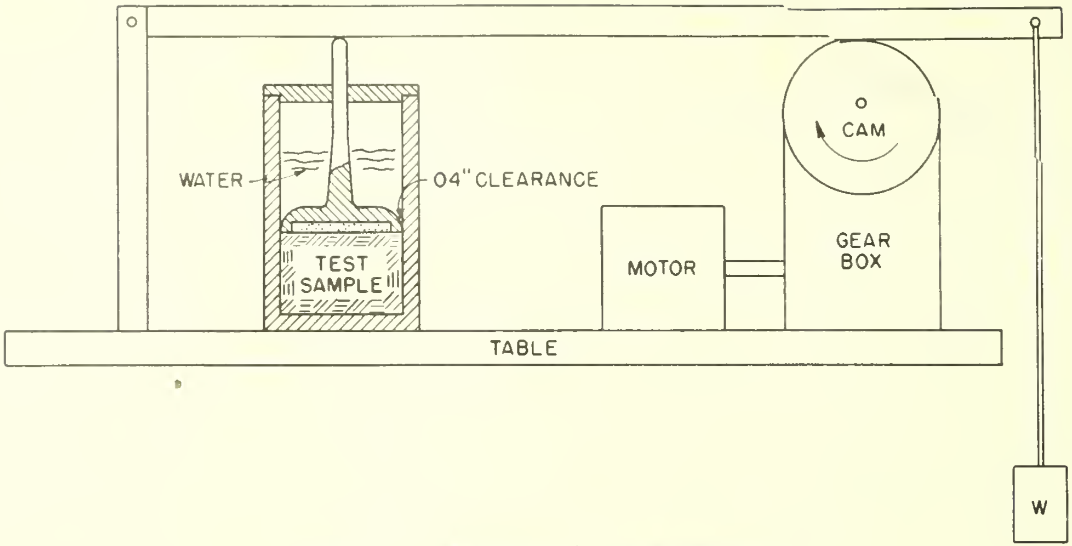
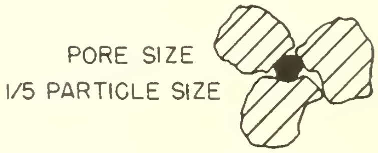


Figure 23.—Apparatus for repetitive loading.



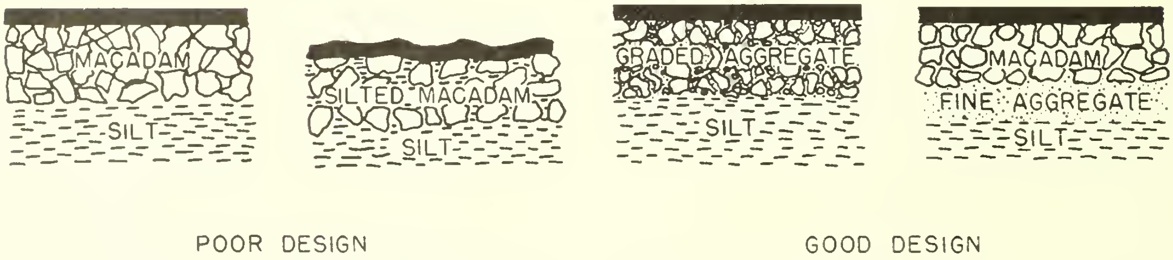


Figure 24.—Intrusion of silt into open aggregate.

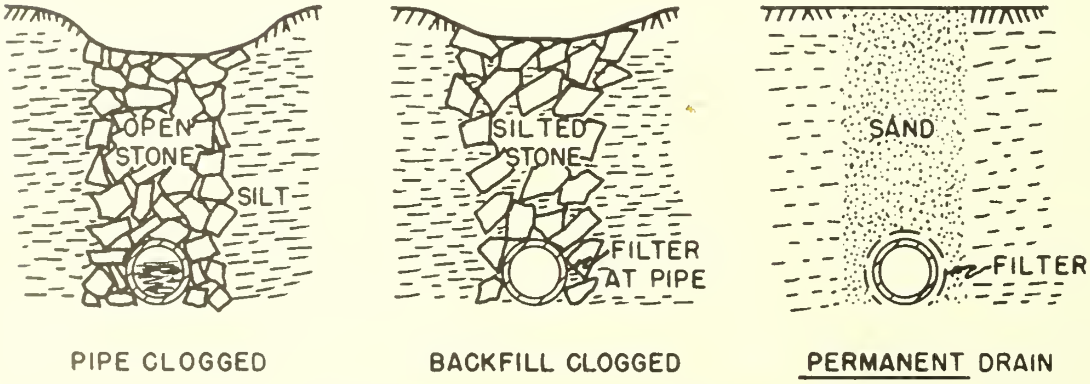


Figure 25.—Intrusion of silt into drains.

the soil can hold back the coarser particles of the soil, infiltration or clogging will not be sufficient to materially impede the drainage. Thus, as shown on the right in figure 25, replacing coarse gravel with sand as backfill in drainage trenches in silt soils prevents the otherwise inevitable intrusion of silt into the gravel. The sand may be kept out of the pipe by putting gravel around open joints or by using perforated pipe with the perforations down.

Granular filters have been used for many years in water filtration and dam drainage. The criterion suggested by Terzaghi and tested and adopted by the Corps of Engineers to prevent intrusion in drain backfills (*10*) is to require that the piping ratio of 15-percent size of filter material to 85-percent size of material to be drained be equal to or less than 5. For plastic clays with sand or silt partings (extremely thin seams) the 15-percent size of the filter—the size than which 15 percent by weight is finer—should be compared to the 85-per-

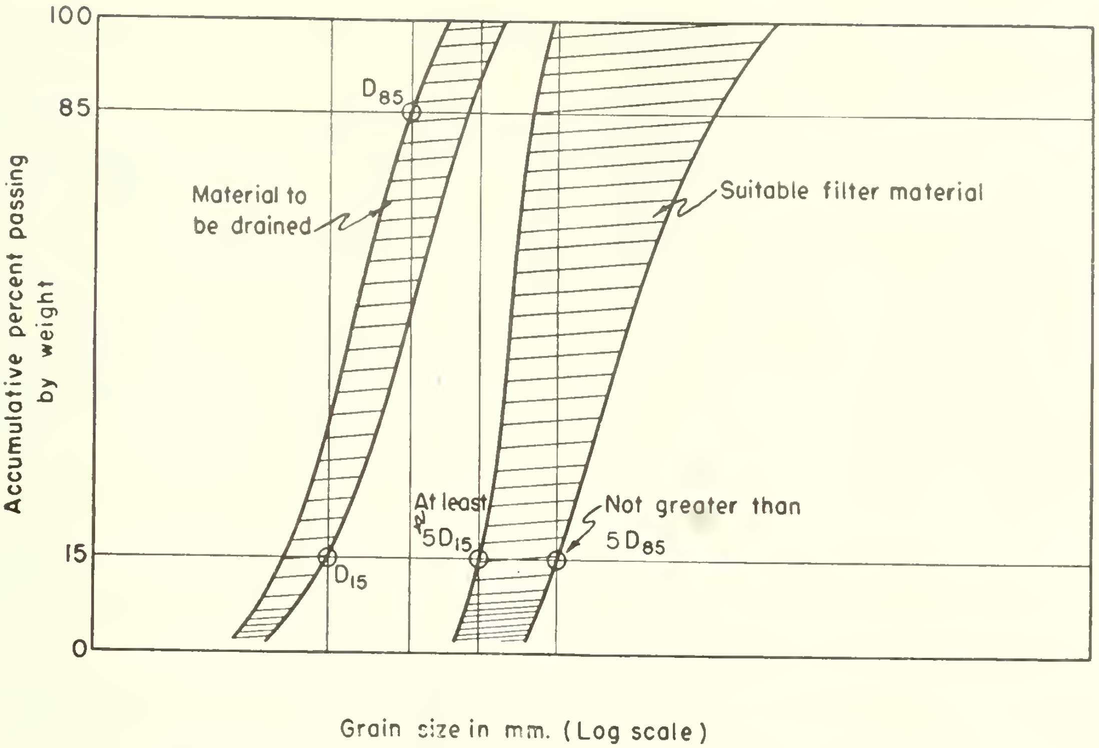


Figure 26.—Specification for grain size of material suitable for filter.

cent size of the sand or silt. For fractured clays without partings, the 15-percent size of the filter need not be less than 1 millimeter regardless of a smaller value indicated by the piping ratio, because the cohesion of the clay will withstand the seepage forces.

To keep the filter material out of the pipe, the Corps of Engineers requires that the ratio of 85-percent size of filter to pipe opening (perforation or slot) be equal to or greater than 2, which may indicate the desirability of two layers of filter material. To insure adequate permeability in the filter, it is required that the ratio of 15-percent size of filter to 15-percent size of material to be drained be equal to or greater than 5. The application of this specification is shown in figure 26. The Connecticut State Highway Department uses higher piping ratios which increase with the uniformity coefficient of the soil (*13*).

The exact ratio permissible between subgrade, subbase, and base courses requires further study. The above specification may be satisfactory except that the minimum of 1 millimeter would not apply.

Density of Base Courses

Besides being required for shear strength, densification of base courses is necessary to minimize traffic consolidation which could cause faulting of joints in rigid pavements or uneven surfaces in flexible pavements. It is a problem, when using open-graded materials, to obtain adequate densification with available equipment. By using a homogeneous test sample in the apparatus shown in figure 23, the densification of materials under repeated loadings has been determined. Table 12 shows the results of such a test on gravel mixtures. The higher initial densities and smaller reductions of thickness under load repetitions is one reason for the use of dense-graded materials for base courses. While the 25-percent admixture gives the least traffic compaction, it has been found necessary to limit the percentage passing the No. 200 sieve to 15 percent and require a plasticity index of not over 6 to prevent softening under wet conditions.

In selecting material for bases under concrete pavements, to prevent pumping, consideration must be given to having enough fines to prevent intrusion of the subgrade soil while not having so much as to cause pumping of the base material itself.

While there is some difference of opinion concerning the need for and the measures of obtaining base drainage, it is generally agreed that thorough compaction is necessary for all types of base material.

Table 12.—*Compaction of gravel mixtures under repetitive loading*

Material, and percentage passing No. 200 sieve	Dry density after—			Thickness reduction—static load to 10,000 repetitions
	Static load, 16 kips per sq. ft.	100 repetitions, 4 kips per sq. ft.	10,000 repetitions, 4 kips per sq. ft.	
	Lb./cu. ft.	*Lb./cu. ft.*	*Lb./cu. ft.*	*Percent*
Graded gravel passing ¾-inch sieve:				
None passing No. 200	112	114	123	9.1
Gravel plus silica dust:				
5 percent passing	117	118	127	7.7
10 percent passing	126	129	136	7.9
15 percent passing	130	134	141	7.9
25 percent passing	130	135	140	5.7
Gravel plus Tuxedo clay:				
5 percent passing	123	125	133	7.3
10 percent passing	130	131	[1] 135	5.8
15 percent passing	130	131	[1] 132	3.7
25 percent passing	130	130	[1] 130	2.1

[1] Density of material below piston; a small amount of pumping occurred.

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U.S. GOVERNMENT PRINTING OFFICE: 1951-951447

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF DECEMBER 31, 1951

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED. CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$18,907	$20,936	$11,024	348.7	$4,998	$2,372	98.9	$18,948	$9,516	370.1	$44,882	$22,912	817.7
Arizona	6,279	2,221	1,591	43.8	532	266	.3	7,824	5,409	120.6	10,577	7,266	164.7
Arkansas	8,596	8,336	4,544	328.0	6,867	3,396	226.4	11,991	6,190	322.2	27,194	14,130	876.6
California	24,327	18,871	9,529	91.0	2,838	1,581	13.8	69,747	33,123	260.2	91,456	44,233	365.0
Colorado	9,109	3,698	2,086	88.2	1,422	805	43.6	11,376	6,056	201.7	16,496	8,947	333.5
Connecticut	7,464	3,350	1,725	11.7	1,427	710	5.1	11,705	6,099	15.3	16,482	8,534	32.1
Delaware	3,234	3,563	1,866	16.7	625	317	18.3	2,706	1,347	19.6	6,894	3,530	54.6
Florida	10,349	11,139	5,762	218.1	7,382	3,808	162.1	16,394	8,200	297.6	34,915	17,770	677.8
Georgia	13,643	12,799	6,546	319.0	11,545	5,266	117.2	27,756	13,896	426.8	52,100	25,708	863.0
Idaho	7,639	10,801	6,752	333.0	1,153	703	36.1	3,841	2,487	84.8	15,795	9,942	453.9
Illinois	37,404	28,396	15,980	207.3	17,273	8,725	141.6	69,683	35,144	587.7	115,352	59,849	936.6
Indiana	22,595	27,953	13,623	144.6	8,965	4,751	70.9	17,709	9,023	191.7	54,627	27,397	407.2
Iowa	10,622	10,411	5,613	307.8	5,895	2,966	183.8	14,114	7,024	585.9	30,420	15,603	1,077.5
Kansas	13,997	8,516	4,038	1,152.8	4,789	2,806	340.8	15,686	7,823	596.8	28,991	14,667	2,090.4
Kentucky	9,723	16,257	8,522	229.6	6,973	3,561	203.4	9,866	4,919	129.5	33,096	17,002	562.5
Louisiana	9,067	16,046	7,673	76.7	8,913	4,153	79.6	17,473	8,757	131.7	42,432	20,583	288.0
Maine	4,986	6,061	3,436	32.6	1,198	567	8.6	7,771	3,934	71.1	15,030	7,937	112.3
Maryland	9,485	1,797	815	22.1	3,351	2,148	3.4	9,241	4,221	25.0	14,389	7,184	50.5
Massachusetts	9,037	10,136	4,349	18.6	2,190	894	6.8	60,474	29,269	50.8	72,800	34,512	76.2
Michigan	17,562	20,166	10,091	385.3	8,457	4,163	141.0	48,233	20,579	247.9	76,856	34,833	774.2
Minnesota	15,287	6,481	3,436	827.8	1,245	607	133.2	21,379	11,678	690.3	29,105	15,721	1,651.3
Mississippi	12,537	8,721	4,487	330.4	3,414	1,678	80.4	17,526	9,117	447.5	29,661	15,282	858.3
Missouri	19,711	24,227	12,462	680.3	11,913	5,872	236.0	34,810	18,328	620.8	70,950	36,662	1,537.1
Montana	11,821	10,931	6,555	361.1	4,129	2,710	59.9	15,018	8,843	231.7	30,078	18,108	652.7
Nebraska	13,296	14,245	7,422	534.3	5,819	2,874	120.3	15,764	7,820	511.1	35,828	18,116	1,165.7
Nevada	7,134	3,932	2,987	113.4	745	624	14.6	3,286	2,721	133.1	7,963	6,332	261.1
New Hampshire	3,803	2,325	1,348	12.2	1,297	646	5.5	5,030	2,505	34.8	8,652	4,499	52.5
New Jersey	10,743	7,073	3,417	16.7	5,513	2,756	4.2	24,414	11,883	18.6	37,000	18,056	39.5
New Mexico	7,165	2,241	1,434	97.0	3,311	2,129	69.2	10,579	6,765	196.3	16,131	10,328	362.5
New York	52,635	73,042	38,359	152.2	20,447	9,308	122.1	89,900	41,191	297.2	183,389	88,858	571.5
North Carolina	11,410	22,226	11,028	538.9	2,569	1,264	60.4	19,867	9,989	369.7	44,662	22,281	969.0
North Dakota	6,964	6,635	3,442	1,047.0	4,299	2,139	448.3	7,703	3,887	640.7	18,637	9,468	2,136.0
Ohio	26,273	22,442	11,080	169.7	18,989	9,478	91.5	69,969	35,464	255.9	111,400	56,022	517.1
Oklahoma	10,931	10,349	5,857	187.4	6,378	3,333	74.1	16,539	8,898	163.6	33,266	18,088	425.1
Oregon	4,394	4,725	2,797	42.6	4,036	2,356	89.0	13,260	7,256	116.5	22,021	12,409	248.1
Pennsylvania	27,171	18,923	9,462	29.6	16,879	8,412	27.2	59,878	29,707	175.5	95,680	47,581	232.3
Rhode Island	4,273	4,639	2,319	36.9	906	453	7.0	14,989	7,620	12.5	20,534	10,392	56.4
South Carolina	6,575	12,136	6,634	390.6	2,041	963	115.2	11,882	5,966	174.4	26,059	13,563	680.2
South Dakota	6,743	3,750	2,170	362.7	2,723	1,652	208.8	12,321	7,113	676.8	18,794	10,935	1,248.3
Tennessee	12,184	9,365	4,359	301.8	9,265	4,305	124.9	24,832	11,837	291.4	43,462	20,501	718.1
Texas	31,101	4,884	2,319	143.4	11,865	6,031	351.2	48,677	25,396	422.7	65,426	33,746	917.3
Utah	5,710	3,645	2,763	86.2	2,979	2,175	40.7	4,158	3,052	60.1	10.782	7,990	187.0
Vermont	2,053	5,447	2,698	61.8	395	202	4.3	5,514	2,738	36.1	11,356	5,638	102.2
Virginia	12,543	17,340	8,594	381.9	5,531	2,795	117.2	20,018	9,979	332.0	42,889	21,368	831.1
Washington	9,108	8,388	3,430	129.5	2,575	1,213	13.6	17,909	9,287	101.8	28,872	13,930	244.9
West Virginia	7,989	9,806	4,947	106.0	4,302	2,162	53.4	10,143	5,057	111.7	24,251	12,166	271.1
Wisconsin	14,103	23,595	12,670	375.9	6,208	3,109	92.5	17,243	8,602	401.4	47,046	24,381	869.8
Wyoming	4,764	460	393	3.4	1,706	1,113	24.2	5,727	3,703	134.5	7,893	5,209	162.1
Hawaii	3,064	5,278	2,341	7.3	2,365	1,102	3.3	9,998	3,879	34.0	17,641	7,322	44.6
District of Columbia	6,388	2,894	1,447	.7				3,592	1,989	1.6	6,486	3,436	2.3
Puerto Rico	6,325	10,161	4,735	45.0	1,205	528	4.4	10,552	4,860	36.4	21,918	10,123	85.8
TOTAL	628,223	601,759	312,957	11,949.3	271,842	137,947	4,698.3	1,095,015	550,146	12,467.7	1,968,616	1,001,050	29,115.3

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